Scuola Universitaria Superiore IUSS Pavia

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Simplified Frameworks and Tools for the Preliminary Assessment and Prioritization of Integrated Structural and Non-Structural Element Seismic Upgrades

A Thesis Submitted in Partial Fulfilment of the Requirements
for the Degree of Doctor of Philosophy in

EARTHQUAKE ENGINEERING AND

ENGINEERING SEISMOLOGY

Obtained in the framework of the Doctoral Programme in
Understanding and Managing Extremes

by

Alessandra Miliziano

November, 2023

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ABSTRACT

Recent earthquakes have demonstrated that buildings' performance and functionality may be significantly reduced by the vulnerability of non-structural elements (NSEs). Damage to building architectural components, utility systems and contents could pose a safety risk and result in substantial monetary and functional loss. A strategy to improve the overall seismic performance of a building consists of integrating seismic upgrades to both structural and non-structural elements. However, designing a viable combination of structural and non-structural upgrades to optimize the upgrade investment may be challenging. This is because the performance of structural and non-structural elements is not independent. The seismic response of a structure represents the seismic demand on its non-structural elements. Therefore, the benefit of a structural upgrade may be reduced due to its impact on non-structural losses. On the other hand, a poor structural performance may void the benefit of a non-structural upgrade investment.

This thesis proposes two frameworks for the preliminary assessment of structural and non-structural upgrades for application in different stages of a design/retrofit project when various levels of information and resources are available. The first framework, named "non-structural upgrade assessment framework", is developed to assess non-structural upgrades using sequential steps which require different levels of sophistication of the input data. The second framework is a pushover-based framework to assess multiple combinations of structural and non-structural upgrades with a computational effort compatible with the limited resources available in a preliminary design phase. From the application of the two frameworks to steel moment-resisting frame archetype buildings, the proposed frameworks were found to be practical and efficient in the preliminary phase of the decision-making process to identify the key drivers that affect non-structural upgrade impact on seismic loss reduction, prioritize non-structural element upgrades and identify viable combinations of structural and non-structural upgrade strategies. The two proposed frameworks are intended as simplified procedures that can help harmonize the seismic performance of structural and non-structural elements and enhance the transparency of the process for identifying viable combinations of structural and non-structural upgrades. As part of this thesis, an Excel tool was also developed to facilitate the implementation of the non-structural upgrade assessment framework.

ABSTRACT (IT)

Terremoti recenti hanno dimostrato che le prestazioni sismiche e la funzionalità di un edificio possono essere significativamente ridotte a causa della vulnerabilità sismica degli elementi non-strutturali. Il danneggiamento degli elementi non-strutturali durante un terremoto può rappresentare un rischio per la sicurezza e comportare sostanziali perdite economiche. Una strategia per migliorare le prestazioni sismiche complessive di un edificio consiste nell'integrare interventi di miglioramento sismico strutturale con interventi di miglioramento sismico non-strutturale. Tuttavia, ottimizzare un investimento per il miglioramento sismico di un edificio combinando interventi strutturali e non-strutturali può essere complesso perché le prestazioni sismiche degli elementi strutturali e non strutturali non sono indipendenti. In particolare, la risposta sismica di una struttura rappresenta la domanda sismica sui suoi elementi non-strutturali. Pertanto, il beneficio di un miglioramento strutturale potrebbe essere ridotto a causa di un suo eventuale impatto negativo sulla prestazione degli elementi non-strutturali. Allo stesso tempo, una scarsa prestazione strutturale potrebbe vanificare il beneficio di un investimento per il miglioramento del comportamento sismico di elementi non-strutturali.

Questa tesi propone due procedure per la valutazione preliminare di interventi di miglioramento sismico strutturale e non-strutturale. Le procedure proposte possono essere applicate utilizzando diversi livelli di dati iniziali che corrispondono alle diverse fasi di un progetto per il miglioramento sismico di un edificio. La prima procedura, denominata "Procedura per la valutazione di interventi di miglioramento sismico non-strutturale", è composta da quattro fasi successive e può essere utilizzata per esaminare diversi interventi di miglioramento sismico di elementi non-strutturali. La seconda procedura, invece, può essere utilizzata per la valutazione preliminare di molteplici combinazioni di interventi di miglioramento strutturale e non-strutturale. L'applicazione delle due procedure proposte ad edifici archetipo ha permesso di dimostrarne l'efficacia. Le procedure proposte possono infatti essere utilizzate per identificare i fattori chiave che influenzano l'impatto di interventi di miglioramento sismico non-strutturale sulle perdite economiche totali di un edificio, classificare in maniera rapida gli interventi di miglioramento sismico di elementi non-strutturali ed identificare combinazioni di interventi di miglioramento sismico strutturale e non-strutturale che possano ottimizzare l'investimento. Le due procedure proposte possono essere utilizzate al fine di armonizzare le prestazioni sismiche degli elementi

strutturali e non strutturali ed identificare in maniera semplice e immediata combinazioni di interventi di miglioramento strutturale e non strutturale efficaci. Nell'ambito di questa tesi è stato sviluppato anche un tool in Excel per facilitare l'implementazione della procedura per la valutazione di interventi di miglioramento sismico non-strutturale.

ACKNOWLEDGEMENTS

This thesis was developed at IUSS Pavia as part of the project "Dipartimenti di Eccellenza," which was funded by the Italian Ministry of Education, University, and Research.

First, I would like to thank my supervisor, Prof. Andrè Filitrault, for his continuous guidance, help, and support during the period of my PhD. His expertise and experience, as well as his encouragement, were essential for the development of my PhD. I'd also like to thank him for his incredible help during my visiting period in Canada. Working on my PhD under his supervision has been an incredible opportunity and privilege.

I would also like to thank my co-supervisors, Prof. Lydell Wiebe, Prof. Stefano Pampanin and Prof. Daniele Perrone for their help, support and suggestions that made this thesis possible. I am extremely thankful for the opportunity to work with them. I'd like to add a special thank to Prof. Wiebe for the opportunity of conducting my visiting period at McMaster University as part of his research group, and for his help and encouragement during my visiting period in Canada and throughout all my PhD.

I would like to acknowledge all the "Non-structural Element Research Group" at IUSS Pavia. A special thanks to Bryan Chalarca and to Paul Steneker for the help in conducting this research. Also, a big thanks to Derek and Milena for their friendship and support. I would also like to thank Livio, Simone, Michele and Giada from the "Sapienza" research group for their friendship and help during my period in Rome. I'd like to thank Karim, Deniz, Moe, Rita and all of my new friends in Pavia, as well as Luisa, Elena, Tommaso, Andrea, Luca, Chiara and all of my friends that have always been there for me. Also, a special thanks to Niccolò for his support, which was extremely important during my PhD.

Finally, I'd like to thank all my family. A particular thanks to my cousins, Valeria and Laura, my aunt Marisa and my uncle Vincenzo for always supporting me. A special thanks to my parents, my sisters, Roberta and Daniela, and to Stella. None of this would have been possible without your help, love, and support.

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a_i^* = Median peak floor acceleration

AM = Amortization conversion factor

C_L = Linear viscous damping constant

$G(x\lfloor y)$ = Conditional complementary cumulative distribution function of X exceeding a specified value x given $Y = y$

H_{ai} = Acceleration corrector factor

$\hat{k}_0^n$ = Stiffness of the fictitious springs that yields to a fundamental period of a fictitiously braced structure equal to $\widehat{T_1}$

$\hat{k}_{0tr}^n$ = Initial trial value of the stiffness coefficient of the fictitious spring that yields to a fundamental period of a fictitiously braced structure equal to $\widehat{T_1}$

N = Number of storeys

PF_1 = Modal participation factor for the first natural mode

r = Internal rate of return

S = Strength ratio

S_a = Spectral acceleration

S_d = Spectral displacement

t = Expected occupancy time

T_1 = Fundamental period of a building

T_b = Fundamental period of the braced frame

T_u = Fundamental period of the unbraced frame

$\widehat{T_{1,tr}}$ = Fundamental period of the fictitious braced frame with trial fictitious spring constants $\hat{k}_{0tr}^{n}$.

$\widehat{T_1}$ = Required fundamental period of the fictitiously braced frame corresponding to a desired first mode viscous damping ratio ξ_1

V = Base shear.

V_{y1} = Estimated yield strength of the building in first mode response

W = Bulding dead weight plus likely live loads.

W = Total weight

w_i/g = Mass assigned to level i of the building.

Greek symbols

α_1 = Modal mass coefficient for the first natural mode

Δ = Median story drift ratio

Δ_r = Median residual drift ratio

Δ_y. = Median story drift ratio calculated at yield

Δ_{roof} = Roof displacement

ΔL = Maximum potential loss reduction due to the upgrade of a component for a given EDP value

ΔL_{max} = Maximum potential loss reduction due to the upgrade of a component in a given EDP range

$\lambda(x)$ = Mean Annual Frequency of x

ξ_1 = First mode viscous damping ratio

ϕ_{i1} = Amplitude of mode 1 at level i of the building.

Acronyms

ADRS = Acceleration-Displacement Response Spectrum

BCR = Benefit-Cost Ratio

C = Collpase

D = Demolition

DM = Damage Measure

DS = Damage State

DV = Decision Variable

EAL = Expected Annual Loss

EDP = Engineering Demand Parameter

IM = Intensity Measure

MAF = Mean Annual Frequency

MCE = Maximum Considered Earthquake

NC = Non-Collapse

NSD = Not-seismically Designed

NSE = Non-Structural Element

NSU = Not-seismically Upgraded

PBEE = Performance-Based Earthquake Engineering

PFA = Peak Floor Acceleration

PGA = Peak Ground Acceleration

PID = Peak Interstorey Drift

R = Repair

RD = Residual Drift

SD = Seismically Designed

SDOF = Single Degree of Freedom

SU = Seismically Upgraded

UC = Upgrade Cost

URC = Unit repair cost

USGS = United States Geological Survey

1.INTRODUCTION

1.1 MOTIVATION

Recent earthquakes have demonstrated that the harmonization of structural and non-structural performance levels is a key aspect to consider to improve the seismic performance of a building. Non-Structural Elements (NSEs) are defined as every part of the building and all its contents except for the structure, which includes elements from heavy mechanical equipment to bookshelves and piping. Even if a structure performs well during an earthquake, large monetary losses and loss of building functionality may be experienced as a result of damages to NSEs. This is not surprising considering that NSEs represent the major portion of total investment in typical buildings [Miranda and Taghavi, 2003] and that they are often not seismically designed, so they typically get damaged from earthquakes with much lower intensities than the ones required to produce structural damages. However, due to the many types of NSEs and the relationship between structural seismic response and non-structural seismic performance, prioritizing non-structural seismic upgrades and identifying optimal combinations of structural and non-structural upgrades may be challenging.

The performance-based earthquake engineering (PBEE) framework developed by the Pacific Earthquake Engineering Research Center (PEER) [Cornell and Krawinkler, 2000; Miranda and Aslani, 2003; Porter, 2003; Moehle and Deierlein, 2004] captures the relationship between the performance of structural and NSEs. The framework has been implemented in the FEMA P-58 methodology, and allows designers to assess building performance and deagreggate expected losses between different building elements. However, using the FEMA P-58 methodology, it is difficult to optimize an upgrade strategy because multiple combinations of structural and non-structural upgrades should be considered using a trial-and-error approach. Steneker *et al.* [2020] recently developed a general optimization procedure in which a genetic algorithm is used within the PEER-PBEE framework to identify optimal combinations of structural and non-structural upgrades. A simplified method, named the Median Shift Probability (MSP) method, was also proposed by Steneker *et al.* [2022] to rapidly assess the effects of structural upgrades on NSEs by considering the impacts of structural modifications on the seismic demand on NSEs. However, although there are frameworks in the literature that allow determining optimal structural and non-structural upgrade combinations, it can be challenging to understand why a certain combination is selected by the algorithm and how much the selection may be sensitive to changes in the input data. These types of insights could be highly helpful in the decision-making process as they provide a better understanding of the problem's drivers and a better control of the optimization output. Moreover, although all

NSEs in a performance model can be included in an optimization process, only a few non-structural upgrades are likely to have a significant impact on seismic loss reduction. However, using the currently available methodologies, it is not easy to assess non-structural upgrades and narrow the number of structural and non-structural upgrade combinations to investigate without running a full optimization analysis, which is cumbersome to execute and requires detailed information on the structural response.

1.2 Objectives and Thesis Outline

This thesis addresses the problem of enhancing the transparency of the process for identifying viable combinations of structural and non-structural upgrades. Two frameworks are proposed in order to help designers better comprehend the key drivers influencing the impact of non-structural upgrades and assist them in prioritizing non-structural upgrades at the preliminary design stage. The first framework, named “non-structural upgrade assessment framework”, is developed to assess non-structural upgrades in a building performance model. The framework comprises four sequential steps characterized by different levels of sophistication of the input data. The first three steps are used to reduce the number of potential non-structural upgrades by removing upgrades that are not likely to produce a significant improvement to building performance. The fourth step is used to prioritize non-structural upgrades that remain after Step 3, using as a metric a benefit-cost ratio. The concept of DV-EDP functions, which directly relate economic losses (Decision Variable, DV) with structural response parameters (Engineering Demand Parameter, EDP), is applied in the proposed framework at a component level. For each component in a building, DV-EDP loss functions tailored to the seismic design rating of the component are developed and used to rapidly compare the potential impact of each non-structural upgrade and prioritize non-structural upgrades. An Excel tool has also been developed as part of this thesis to facilitate the practical implementation of the tool. The second framework proposed in this thesis allows to combine structural and non-structural upgrades. A simplified procedure is used in the framework to quickly assess the effect of a structural upgrade on the NSE seismic demand and performance, and identify viable integrated structural and non-structural upgrade strategies.

The objective of this thesis is to develop frameworks to increase the transparency of the process for identifying viable combinations of structural and non-structural upgrades, seeking to answer the following questions:

1. How can a designer identify the key drivers that affect non-structural upgrade impacts on seismic loss reduction?
2. Which strategy can be used to recognize when the seismic performance of a building may be significantly affected by a non-structural upgrade and narrow the number of non-structural upgrades to further investigate?

3. What steps may be implemented to quickly prioritize non-structural upgrades that could significantly affect seismic loss reduction?
4. How does the prioritization of non-structural elements change if non-structural upgrades are combined with different structural upgrade strategies?

These questions are addressed and discussed in the chapters outlined below:

Chapter 2 provides a brief overview of key research projects in the area of seismic risk assessment and optimization of structural and non-structural seismic upgrades. The chapter introduces the concept of PBEE and the role of NSEs. A discussion on component-based and storey-based loss estimation approaches is provided and recent optimization frameworks to identify optimal combinations of structural and non-structural upgrades are discussed.

Chapter 3 presents a four-step framework, named "non-structural upgrade assessment framework", to conduct a preliminary assessment and prioritization of non-structural upgrades. Starting with a list of every component in a performance model, each of the four steps is used to exclude a set of components whose upgrade would have negligible impact on the total seismic loss of the building or whose upgrade cost would be excessively high compared to the benefit of the upgrade. The output of the last step is the prioritization of non-structural upgrades. Each step should be applied sequentially and the level of sophistication and input data required in each step depends on the intent and/or the resources available to the analyst in a design/retrofit situation (e.g. preliminary design/proof of concept in the first step to final design/retrofit in the fourth step). This makes the proposed framework general and flexible. The output of each step provides insights into non-structural upgrades that can help control and understand the final output. A brief overview of an Excel tool developed to facilitate the practical implementation of the framework is also provided in this chapter.

Chapter 4 discusses the results from the application of the non-structural upgrade assessment framework developed in Chapter 3 to three case study buildings. Three steel moment resisting frames with three, six, and nine storeys are analyzed. The validation of the framework is performed by comparing the framework output with the results obtained using the rigorous FEMA P-58 methodology (FEMA P-58-1, 2018).

Chapter 5 presents a more general framework that incorporates the non-structural upgrade assessment tool introduced in Chapter 3 to identify viable combinations of structural and non-structural upgrades. An equivalent Single Degree of Freedom (SDOF) approximation is used to perform the structural analysis and component DV-EDP functions tailored to the seismic design rating of each component are used to perform the loss analysis. The framework is intended as a preliminary simplified method for evaluating the impact of various integrated structural and non-structural upgrade strategies in a way that is straightforward and consistent with the limited resources available at the early design stage.

Chapter 6 discusses the application of the general framework developed in Chapter 5 to a six-storey steel moment resisting frame. Two structural upgrade strategies are investigated, which are 1) the use of hysteretic dampers, and 2) the use of linear viscous dampers. Target performance objectives are defined in terms of probability of collapse and expected annual loss. The framework's output, which combines structural and non-structural upgrades to enable the achievement of the target performance objectives, is validated using the FEMA P-58 methodology.

Chapter 7 summarizes the key results from this thesis. From the results, conclusions are drawn and potential future research directions are outlined.

The following three appendices are also included in the thesis:

Appendix A presents four different damage aggregation methods, referred to as edge cases, that can be used to model economies of scale in the FEMA P-58 and in the frameworks proposed in this thesis. An illustrative example is introduced to illustrate the four edge cases for damage aggregation.

Appendix B presents the fragility functions used to model non-structural elements in the archetype building examples discussed in Chapters 4 and 6.

Appendix C contains instructions on how to use the non-structural upgrade assessment Excel tool developed as part of this research.

A schematic representation of the seismic upgrade strategies investigated in each chapter is provided in Figure 1.1.

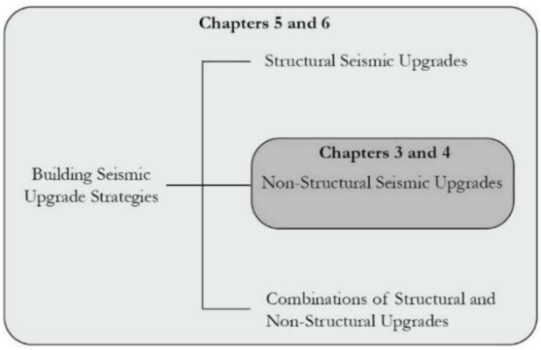

Figure 1.1 Seismic Upgrade Strategies addressed in each chapter.

The framework proposed in Chapters 5 and 6 comprises all possible building upgrade strategies: structural upgrades, non-structural upgrades, and combinations of structural and non-structural upgrades. Chapters 3 and 4 present a framework that focuses only on the prioritization of non-structural upgrades for a given structural configuration. As illustrated in the figure, the framework proposed in Chapters 3 and 4 is incorporated into the more general framework introduced in Chapters 5 and 6.

2. PERFORMANCE-BASED EARTHQUAKE ENGINEERING AND HARMONIZATION OF STRUCTURAL AND NON-STRUCTURAL PERFORMANCE LEVELS

2.1 CHAPTER OVERVIEW

This chapter provides a discussion on the development of performance-based earthquake engineering and the impact of non-structural seismic performance on the overall performance of a building. Component-based and storey-based loss assessment procedures are presented and an overview of recently developed optimization methodologies to combine structural and non-structural upgrades is provided. These provide the starting point to the frameworks that are developed in this thesis.

2.2 PERFORMANCE-BASED EARTHQUAKE ENGINEERING AND THE ROLE OF NON-STRUCTURAL ELEMENTS

Over the last decades, a crucial advancement in the field of earthquake engineering has been the development of the concept of Performance-Based Earthquake Engineering (PBEE). PBEE can be defined as the practice of designing buildings to achieve "predictable performance levels" when subjected to specified earthquake hazard intensities [SEAOC Vision 2000 Committee, 1995]. This concept can be easily visualized using the Performance Design Objective Matrix in Figure 2.1 , in which seismic hazard levels with different return periods are coupled with discrete building performance levels identified as Operational, Immediate Occupancy, Life-Safety, and Collapse-Prevention. These performance levels reflect expectations regarding the level of damage in a building and the consequence of damage following an earthquake.

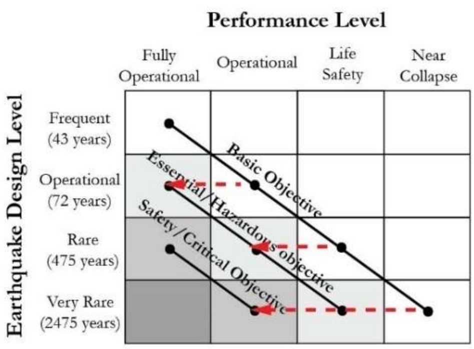

Figure 2.1. Seismic Performance Design Objective Matrix (modified from SEAOC Vision 2000, 1995).

At the onset of PBEE, for an ordinary residential or commercial building, a Life-Safety performance objective was typically accepted for a Design Earthquake level with a return period of approximately 500 years (Basic Objective Curve in Figure 2.1). However, recent earthquakes such as the Canterbury sequence of 2010- 2011 [The Canterbury Earthquake Royal Commission, 2012] have demonstrated that even if a structure achieves the Life-Safety performance objective, excessively large socio-economic losses can be experienced as a result of repair costs, loss of building functionality and demolition. Similar observations demonstrated the need for targeting performance levels beyond Life-Safety and Collapse-Prevention, which is represented in the Performance Design Objective Matrix by a shift of the Basic Objective Curve to the left.

Moving from Collapse Prevention and Life-Safety performance levels to an Operational performance level, the role of Non-Structural Elements (NSEs) became more crucial. In the event that the structure collapses, the non-structural performance would not be critical. However, if the structure performs well during an earthquake, a poor performance of NSEs can compromise the functionality of a building and produce large economic losses. As demonstrated by recent seismic events, losses due to damages to NSEs can even exceed losses due to structural damages (FEMA, 2015; Miranda et al., 2012; EERI, 2012). This is because NSEs represent the major portion of total investment in typical buildings, as illustrated in the iconic image developed by Miranda and Taghavi [2003] (Figure 2.2). Also, damage to NSEs can pose a risk to life safety and severely limit the functionality of critical facilities such as hospitals.

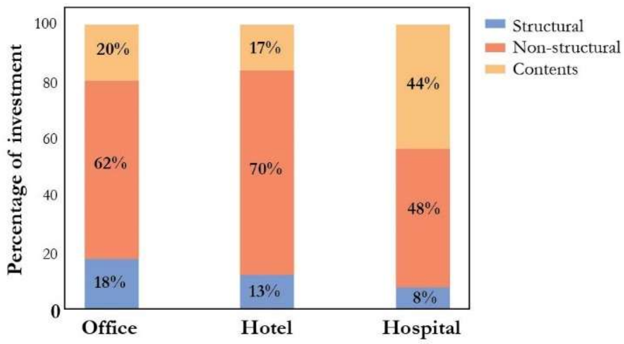

Figure 2.2. Distribution of investments for different building occupancies [Miranda and Taghavi, 2003].

2.3 THE PEER FRAMEWORK AND THE FEMA P-58 METHODOLOGY

A further important development for the seismic design and assessment of buildings is the introduction of new continuous performance measures more meaningful to decisionmakers than the discrete performance levels described in the previous paragraph. Starting in 2001, FEMA initiated the first in a series of projects with the Applied Technology Council, which led in 2012 to the publication of the first version of the FEMA P-58 methodology (FEMA P-58, 2012) for Seismic Performance Assessment of Buildings. In the methodology, performance is expressed in terms of probable damage and resulting consequences associated with earthquakes, explicitly considering uncertainties in performance assessment. Typical performance measures used in the FEMA P-58 methodology are casualties, repair cost, and repair time. These performance measures are more useful in the decision-making process and can be easily communicated to stakeholders.

The technical basis of the FEMA P-58 methodology is the framework for performance-based earthquake engineering developed by researchers at the Pacific Earthquake Engineering Research Center (PEER) (Cornell and Krawinkler, 2000; Miranda and Aslani, 2003; Porter, 2003; Moehle and Deierlein, 2004). It comprises the four main analysis steps illustrated in Figure 2.3: hazard analysis, structural analysis, damage analysis, and loss analysis.

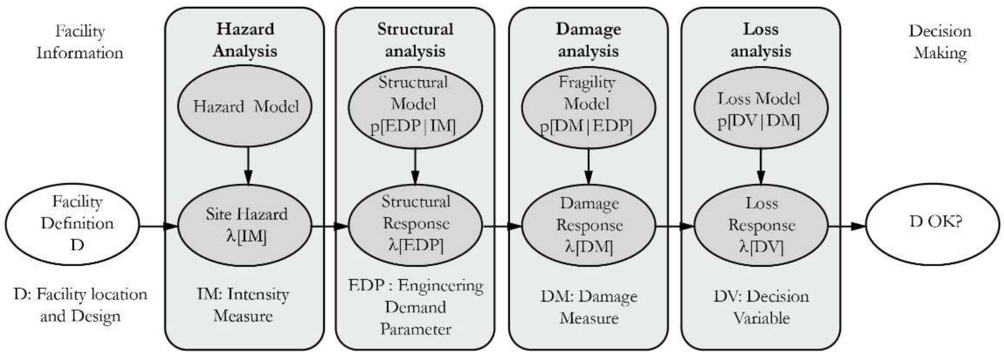

Figure 2.3. Main steps of the PEER framework [Porter, 2003].

The first step is the hazard analysis in which the frequency of exceedance of a ground motion intensity IM is calculated. The typical IM used in this step is the spectral acceleration at a specific period or peak ground acceleration. The output of the hazard analysis step is used in the structural analysis step to analyze a building model and obtain the building response at different seismic hazard levels. The structural response is expressed in terms of Engineering Demand Parameters (EDPs), which correlate well with damage to both structural and non-structural elements. Typical examples of building-relevant EDPs are Peak Interstorey Drifts (PID) and Peak Floor Accelerations (PFA). In the damage analysis, the building response (EDP values) is related to a Damage Measure (DM). DMs express the level of damage experienced by building structural or non-structural elements. The relation between EDPs and DMs is expressed in the form of fragility functions, which describe the probability of exceeding a damage state conditioned on EDP values. Finally, in the loss analysis step, DMs are related to Decision Variables (DVs), which are typically repair cost, repair time, and casualties.

Using the total probability theorem, the Mean Annual Frequency (MAF) of exceedance of a DV can be conceptually calculated through a triple integral given by Equation 2.1:

$$\lambda(DV) = \iiint G(DV|DM) \cdot |dG(DM|EDP)| \cdot |dG(EDP|IM)| \cdot |d\lambda(IM)| \qquad (2.1)$$

Where $\lambda(x)$ is the MAF of x and $G(x|y)$ is the conditional complementary cumulative distribution function (CCDF) of X exceeding a specified value x given $Y = y$. As a closed-form solution of the integral is difficult, the integration is implemented in the FEMA P-58 methodology using Monte Carlo Simulation.

2.4 Simplified Loss Estimation Procedures: The Ramirez and Miranda Approach

The FEMA P-58 methodology represents the current state-of-the-art framework for building-specific seismic risk assessment. However, the level of information on the building inventory (structural and non-structural) and on the structural response required to apply the methodology may be discouraging for practitioners. For this reason, much research has also been dedicated to the development of simplified loss estimation procedures (Bradley et al., 2009; Zareian and Krawinkler, 2012, Welch et al., 2014, Ligabue et al., 2018, Perrone et al.,2019; O'Reilly and Calvi, 2020, Del Vecchio et al., 2020).

An alternative to the FEMA P-58 methodology is the storey-based loss estimation approach proposed by Ramirez and Miranda (Ramirez and Miranda, 2009; Ramirez and Miranda, 2012). In this approach, the loss estimation is not performed at a component level as in the FEMA P-58 but at each storey level of a building. The main simplification of the loss estimation process is the use of storey DV- EDP functions, which directly relate economic losses with structural response parameters, merging the steps of damage and loss analyses (Figure 2.4). For a given EDP_j, the expected loss $E[L_j \mid NC \cap R, EDP_j]$ in component j given that collapse does not occur (non-collapse NC) and the building is repaired (repair R), is expressed as:

$$E\left[L_j \middle| NC \cap R, EDP_j\right] = \sum_{i=1}^{m} E\left[L_j \middle| NC \cap R, DS_i\right] P\left(DS = ds_i \middle| NC \cap R, EDP_j\right) \quad (2.2)$$

where m is the number of damage states in the *jth* component, $E[L_j \mid NC \cap R, DS_i]$ is the expected value of loss in component j when it is in damage state i, and $P(DS = ds_i \mid NC \cap R, EDP_j)$ is the probability of the *jth* component being in damage state i, given that it is subjected to an *EDPj*. In the framework of the storey-based loss estimation approach, storey DV-EDP functions are used to perform the loss estimation. More recently Papadopoulos et al. [2019] proposed a simple methodology for the derivation of storey DV-EDP functions, named storey-loss functions (SLF), compatible with the FEMA P-58 methodology and Shahnazaryan et al. [2021] developed a Python-based toolbox for the development of user-specific and customizable SLFs.

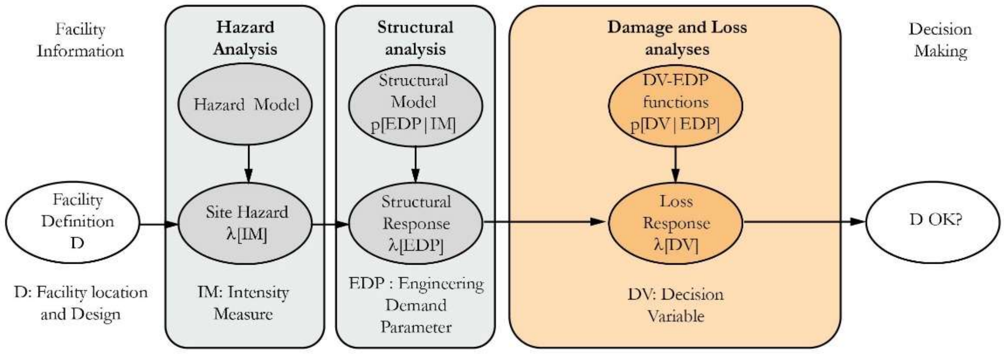

Figure 2.4 PEER framework and DV-EDP Loss Functions.

DV-EDP functions are used to calculate expected economic losses when a building does not collapse and is repairable. These losses need to be combined with the expected losses from collapse and demolition. Using the total probability theorem, the expected value of total economic loss in a building $E[L_T \mid IM]$ conditioned on a ground motion intensity $IM = im$, can be calculated as the weighted sum of expected losses from three mutually exclusive, collectively exhaustive events: collapse does not occur and damage in the building is repaired (i.e., $NC \cap R$); collapse does not occur but the building is demolished and rebuilt (i.e., $NC \cap D$); collapse occurs and the building is rebuilt (i.e. C).

$$E[L_T|IM] = E[L_T|NC \cap R, IM]P(NC \cap R|IM) + E[L_T|NC \cap D]P(NC \cap D|IM) + E[L_T|C]P(C|IM) \tag{2. 3}$$

where:

- $E[L_T \mid NC \cap R_j]$ is the expected value of losses when the building does not collapse and is repaired given an earthquake with a ground motion intensity $IM = im$;

- $E[L_T \mid NC \cap D]$ is the expected value of losses when the building does not collapse but is demolished; and
- $E[L_T \mid C]$ is the expected value of losses when the building collapses.

The weights of these three mutually exclusive events are:

- $P(NC \cap R \mid IM)$, which is the probability that the building does not collapse and is repaired given an earthquake with a ground motion intensity $IM = im$;
- $P(NC \cap D \mid IM)$, which is the probability that the building does not collapse but
- is demolished given an earthquake with a ground motion intensity $IM = im$; and
- $P(C \mid IM)$, which is the probability that the building collapses under a ground motion with a level of intensity $IM = im$.

After the expected losses at different seismic intensity levels are calculated using Equation 2.3, the Expected Annual Loss (EAL) can be computed by integrating expected losses over all considered seismic intensities.

2.5 HARMONIZATION BETWEEN STRUCTURAL AND NON-STRUCTURAL ELEMENTS

The loss assessment procedures introduced in the previous paragraphs can be used to assess the effectiveness of a building seismic upgrade in reducing expected seismic losses. Building seismic upgrades include upgrades to structural elements, upgrades to non-structural elements, and combinations of structural and non-structural upgrades. When designing a building seismic upgrade, it is crucial to consider that structural and non-structural performance are not independent. The seismic response of a structure represents the seismic demand on its NSEs. Therefore, the benefit of a structural upgrade, which modifies the structural response to earthquakes, may be reduced if the change in the structural response produces an increased seismic demand on NSEs. On the other hand, NSE upgrades may not be as effective if a poor structural performance is achieved during an earthquake.

The problem of harmonizing the performance between structural and NSEs represents a challenging area in the field of earthquake engineering, and throughout the past decades, there have been modifications to how this problem is thought about. The traditional way of thinking about seismic risk reduction strategies is to first design a structural upgrade and then assess the performance of the entire building, which includes both structural and NSEs. If the building performance is judged inadequate, modifications are made to the structural upgrade strategy, and thus the seismic demand on NSE is modified while the NSE capacity remains the same. Using this approach, losses to NSEs are controlled indirectly by limiting their seismic demand. During the last decades, many advancements have been made in the field of seismic design of NSEs (FEMA E-74, 2012) and, as a result, increased research interest has focused on strategies that involve simultaneously taking into account upgrades to both structural and non-structural upgrades.

When considering upgrades to both structural and non-structural elements, a key question arises. How can a designer identify the optimal upgrade strategy when combining structural and non-structural upgrades? In the literature, there are examples of studies that compare the cost and benefit of structural upgrades [Galanis et al. 2018; Hofer et al. 2018]. Also, the impact of structural and NSE upgrades on the life-cycle cost of buildings has been investigated by Cardone et al. [2019] and Bianchi et al. [2021]. However, the framework developed recently by Steneker et al. [2020] is the first that aims to identify the optimal combinations of structural and non-structural upgrades. The framework implements a genetic algorithm to systematically identify optimal upgrade strategies within the PEER-PBEE framework. Each "individual" in a potential upgrade generation is represented in the genetic algorithm by a string of bits. Each bit represents a NSE in the building, given a particular structural configuration, and its value is either 0 or 1 depending on whether the NSE has been seismically upgraded. The use of a genetic algorithm allows the identification of optimal combinations of structural and NSE upgrades with a reduced computational time compared to applying the FEMA P-58 methodology using a trial-and-error approach. However, in a preliminary design phase, its application can be onerous and computationally expansive. To simplify the optimization process and provide a tool that can be used in the preliminary stage of decision-making, a three-level framework was proposed by Steneker et al. [2022]. The last level of the framework, which is the most accurate but also the most onerous one, uses a genetic algorithm, while in the first two levels, a simplified procedure named the Median Shift Probability (MSP) method is proposed. Structural upgrades are defined in the MSP method by changes in probability distribution curves describing structural collapse and structural response parameters such as residual drift, peak floor acceleration and storey drift ratio. In the first level of the framework, the changes in the probability distribution curves are based on engineering judgment, while in the second level, analytical models are developed to refine each upgrade curve.

2.6 Review and Discussion

The key research projects in the area of seismic risk assessment of buildings and harmonization between structural and non-structural performance levels were discussed in this chapter. As outlined in the introduction, the problem of enhancing the transparency of the process for identifying viable combinations of structural and non-structural upgrade strategies will be addressed in the next chapters. The concept of DV-EDP functions introduced in this chapter, which serves as the foundation for storey-based loss estimation approaches, will be applied in the next chapter at a component level and used as a basis for developing a framework for the preliminary assessment of non-structural upgrades.

3. NON-STRUCTURAL UPGRADE ASSESSMENT FRAMEWORK

3.1 CHAPTER OVERVIEW

This chapter presents a four-step framework for the assessment and prioritization of non-structural element upgrades. As discussed in Chapters 1 and 2, the objective of the framework is to provide designers with insights into non-structural upgrade impact on seismic loss reduction for different levels of sophistication of the input data. An overview of the non-structural upgrade assessment Excel tool developed as part of this thesis is also provided in the chapter.

3.2 OVERVIEW OF THE PROPOSED FRAMEWORK

As illustrated in Figure 3.1, the framework proposed in this chapter comprises four steps. Each step is applied sequentially: the first three steps are used to eliminate non-structural upgrades that would have a small impact on the building's seismic performance, while the last step is used to prioritize non-structural upgrades that remain after Step 3. The non-structural element upgrade set assessed in the first step of the framework (STEP1 NSE set) includes all non-structural upgrades considered in the performance model, while the non-structural element upgrade sets assessed in the other steps of the framework (STEP2 NSE set, STEP3 NSE set and STEP4 NSE set) include only non-structural upgrades that remain after the previous step is run.

Input data with an increasing sophistication level are required from Step 1 to Step 4:

Step 1 requires general input data on the building, such as number of floors, building occupancy, and floor area. The non-structural elements to be included in the performance model should also be specified.

Step 2 requires as additional input an estimate of non-structural upgrade costs.

Step 3 needs as additional input the maximum expected values of inter-storey drift and peak floor acceleration for each level of the the investigated building.

Step 4 is the most refined and requires the following additional input data: the hazard curve at the site, structural response parameters for different earthquake intensities (average EDP values and collapse fragility), and parameters to perform the benefit-cost analysis (i.e. expected occupancy time of the building, internal rate of return).

Depending on the intent and/or resources available to the analyst, only the first preliminary design/proof of concept steps or all the steps until the final non-structural upgrade prioritization may be run. Also, the level of sophistication of the structural analysis type to obtain the input data required for Step 4 depends on the intent of the designer, and time history analyses or simplified analyses may be used.

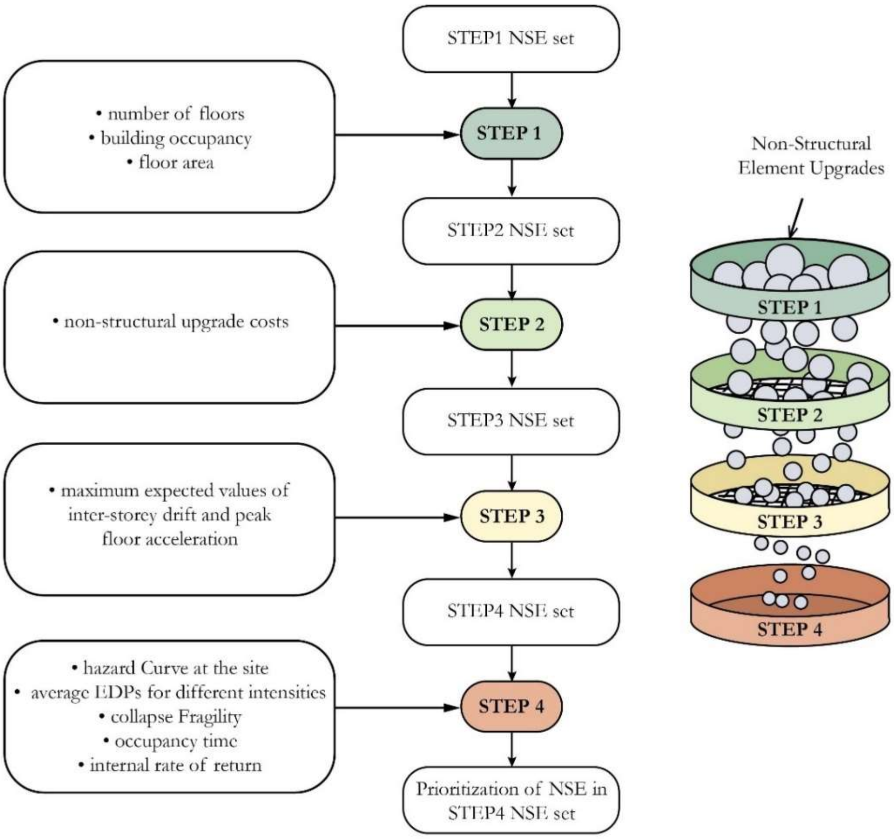

Figure 3.1 Input data required for eah step of the proposed framework.

3.3 DEVELOPMENT OF COMPONENT LOSS-EDP FUNCTIONS

As discussed in Chapter 1, although a wide range of non-structural elements is typically included in a building performance model, not all non-structural upgrades are likely to have a significant impact on seismic loss reduction. However, in the context of the PBEE framework, it is difficult to compare the potential impact of non-structural upgrades on seismic loss reduction without running a full loss estimation analysis. This is because the potential impact of each non-structural upgrade depends on several factors including fragility functions and consequence functions in the not-seismically upgraded and seismically upgraded configuration, number of damage states, and component quantity. In the storey-based loss estimation approach proposed by Ramirez and Miranda (Ramirez and Miranda, 2009), for each storey, the information on structural and non-structural element fragility functions, consequence functions, number of damage states, and quantities are combined to obtain storey-loss functions that directly relate EDP values with storey losses. The use of storey DV-EDP functions simplifies the loss estimation process as it allows to easily compute expected losses conditioned on structural response parameters. However, the loss estimation is performed at a storey level, without distinguishing between the contribution of each element. In the proposed framework, the concept of DV-EDP functions is applied at a component level. For each non-structural element, two Loss-EDP functions corresponding to the element's seismically upgraded and not-seismically upgraded configurations are developed to directly relate EDP values with expected seismic losses.The Loss-EDP functions are used to perform the framework steps in Figure 3.1 and should be developed by users of the framework. If the Excel tool included with this thesis is used, the Loss-EDP functions of each non-structural upgrade are automatically generated and used to perform each step of the framework. Figure 3.2 illustrates the process to develop the Loss-EDP function of a component *j* located on floor *f*. After estimating the element quantity q_j on floor *f*, for each EDP_k within the range of considered EDP values, the steps listed below should be followed:

i) From the element fragility functions, calculate the probability of the element being in each damage state ds_i, given that the structure does not collapse, it is repaired and it is subjected to EDP_k, $P(DS = ds_i|NC \cap R, EDP_k)$;

ii) Using Equation 3.1, calculate the quantity of damaged items $q_{j,dsi}$ in each damage state:

$$q_{j,ds_i} = q_j \cdot P(DS = ds_i|NC \cap R, EDP_k) \tag{3. 1}$$

iii) Use the quantity of damaged items $q_{j,dsi}$ as input in the consequence functions to estimate component unit repair cost URC_i corresponding to each damage state;

iv) Use Equation 3.2 to calculate the expected loss given that the structure does not collapse, it is repaired and it is subjected to EDP_k:

$$E[L_j|NC \cap R, EDP_k] = \sum_{i=1}^{m} E[L_j|NC \cap R, DS_i]P(DS = ds_i|NC \cap R, EDP_k) \tag{3. 2}$$

where m is the number of damage states in the *jth* element, and $E[L_j|NC \cap R, DSi]$ is the expected value of loss for the *jth* element when it is in damage state DS_i, given that the structure does not collapse and it is repaired. The expected loss $E[L_j|NC\cap R, DS_i]$ can be calculated using Equation 3.3 as the product of the quantity of damaged items in DSi and the unit repair URC_i of damage state DS_i.

$$E[L_j|NC \cap R, DS_i] = URC_i \cdot q_{j,ds_i} \tag{3. 3}$$

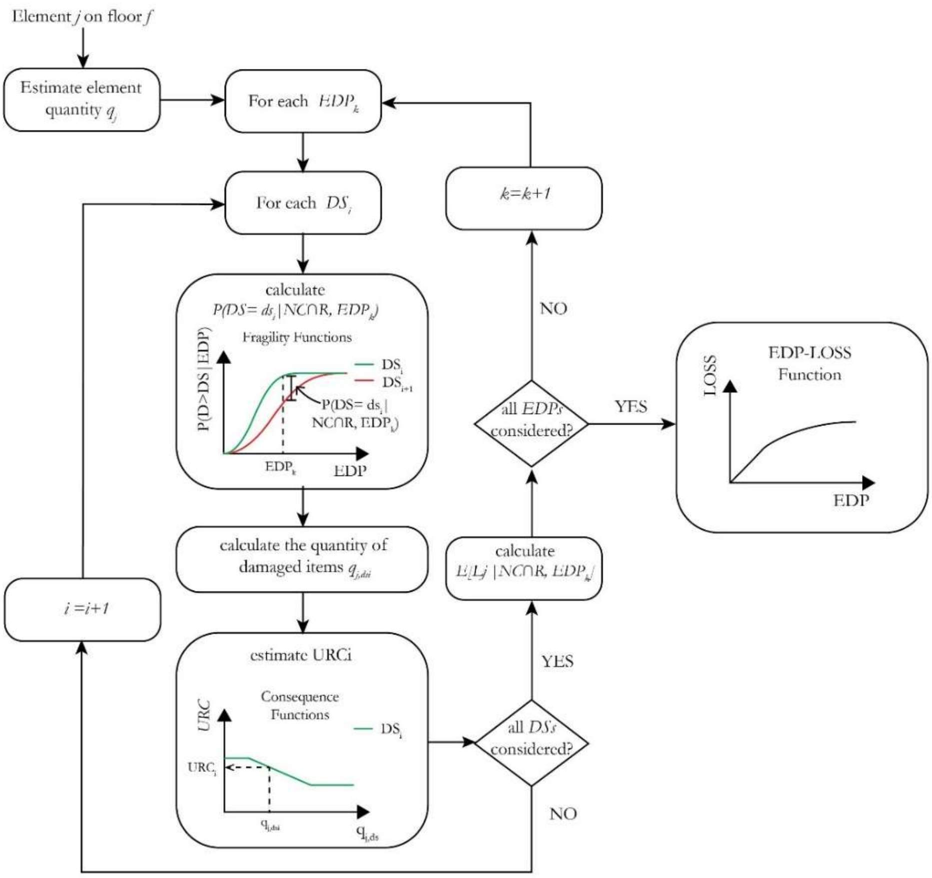

Figure 3.2 Key steps for developing component Loss-EDP functions.

To simplify the loss estimation process, the uncertainty in repair cost and in the number of damaged units is neglected in the framework and an average unit repair cost $URCi$ is used to calculate the $E[Lj|NC, DSi]$. The damage quantity input in the consequence function to estimate the unit repair cost URC depends on how damages are aggregated to account for economies of scale. In general, four different damage aggregation methods can be used to model economies of scale [Banihashemi *et al.,* 2022]: 1) aggregate damage across all floors and from every damage state; 2) aggregate damage across all floors but only from one damage state of interest at a time; 3) aggregate damage on the floor of interest from every damage state; and 4) aggregate damage only on one floor and from one damage state of interest at a time. An illustrative example is presented in Appendix A to illustrate the four damage aggregation edge cases. For the purpose of this framework, it is assumed that damage is aggregated only on one floor and from one damage state of interest at a time. Therefore, the damage quantity on the floor of interest from one damage state is used as input in the consequence function to estimate unit repair cost corresponding to that damage state.

3.4 DISCUSSION OF FRAMEWORK STEPS

This paragraph describes the parameters used to remove non-structural element upgrades in the first three steps of the framework and prioritize NSE upgrades that remain after Step 3. An overview of the four steps of the framework is provided in Figure 3.4.

Step 1. The first step requires the lowest amount and level of sophistication of input data, and it can be run without performing any structural analysis. The objective of this step is to remove non-structural elements whose upgrades are unlikely to provide a significant seismic loss reduction independently from the structural response. These elements could be, for example, NSEs with a very low repair cost compared to other NSEs in the building or elements whose performance is only marginally improved by their seismic upgrades. When looking at the Loss-EDP functions of such elements, a small difference between the element Loss-EDP functions in the not-seismically upgraded (NSU) and seismically upgraded (SU) configurations can be noticed. If the two element Loss-EDP functions are very close in the EDP range of interest, the element seismic upgrade is not likely to produce a large loss reduction.

To perform Step 1, an EDP range of engineering interest is selected and, for each EDP_k in the considered range, the loss reduction $\Delta L_{j,k}$ due to the upgrade of an element j is calculated (see Equation 3.4 and Figure 3.3):

$$\Delta L_{j,k} = E\left[L_{j,NSU} | NC \cap R, EDP_k\right] - E\left[L_{j,SU} \; | NC \cap R, EDP_k\right] \qquad (3.\ 4)$$

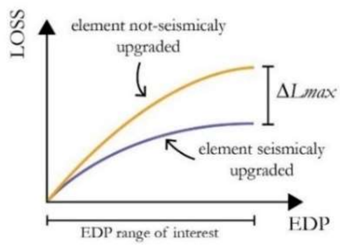

Figure 3.3 Element EDP-Loss functions and ΔL_{max} parameter.

The maximum loss reduction $\Delta L_{j,max}$ in the EDP range of interest is used as a parameter to exclude the first set of non-structural upgrades. If $\Delta L_{j,max}$ is below a threshold specified by the designer, the non-structural element upgrade is removed from the set of potential non-structural upgrades. If $\Delta L_{j,max}$ is above the threshold, the non-structural upgrade is kept in the set of upgrades to be investigated in Step 2 of the framework. $\Delta L_{j,max}$ represents the potential seismic loss reduction due to the considered non-structural upgrade for the earthquake scenario (i.e. EDP value) that maximizes the benefit of the upgrade. The actual loss reduction produced by the upgrade cannot exceed this maximum potential value. However, depending on the structural response, a seismic loss reduction much lower than the maximum potential value may be achieved. For example, if the maximum potential loss reduction occurs for an EDP value that corresponds to an earthquake intensity with a high probability of building collapse, the actual impact of the non-structural element upgrade would be much lower. For this reason, $\Delta L_{j,max}$ is not used to prioritize non-structural upgrades, but only to exclude non-structural element upgrades whose impact would be negligible even for an earthquake scenario that maximizes their benefits.

Step 2. The additional input data required in Step 2 is the upgrade cost of each non-structural element remaining from Step 1, UC_j. In the second step, the ratio between the maximum potential loss reduction $\Delta L_{j,max}$ calculated in Step 1 and the upgrade cost UC_j is used as a parameter to exclude non-structural upgrades. If the ratio $\Delta L_{j,max}/UC_j$ of a non-structural upgrade is lower than a given threshold specified by the designer, the upgrade is excluded from the set of potential non-structural upgrades to be considered in Step 3 of the framework. In general, a value of $\Delta L_{j,max}/UC_j$ smaller than 1 indicates that the cost of the upgrade is larger than the maximum potential loss reduction produced by the upgrade. Therefore, for small values of $\Delta L_{j,max}/UC_j$, a non-structural upgrade is not likely to have a large benefit-cost ratio. Similarly to the paramer used in Step 1, the ratio $\Delta L_{j,max}/UC_j$ can be used to exclude non-structural upgrades but not to prioritize them as, depending on the structural response, the actual ratio between the seismic loss reduction produced by the upgrade and the upgrade cost may be much lower than the maximum potential one.

Step 3. Step 3 requires as additional input the maximum expected EDP values for the investigated building. These maximum EDP values can be defined as the EDPs corresponding to a specified earthquake intensity level or to a certain probability of building collapse. Depending on the intent of the analyst, engineering judgment or more refined analyses can be used to estimate maximum expected EDP values. Based on the input maximum expected EDPs, the EDP range of interest assumed in Step 1 is modified and, for each non-structural upgrade, $\Delta L_{j,max}$ and $\Delta L_{j,max}/UC_j$ corresponding to the new EDP range are calculated. If $\Delta L_{j,max}$ or $\Delta L_{j,max}/UC_j$ are below the threshold specified by the analyst, the considered upgrade is excluded from the set of potential non-structural upgrades.

Step 4. Non-structural elements that pass Step 3 are now prioritized in Step 4. The upgrade benefit-cost ratio BCR_j is used as a metric to perform the prioritization:

$$BCR_j = \frac{EAL_{j,NSU} - EAL_{j,SU}}{UC_j} AM \tag{3. 5}$$

where $EAL_{j,NSU}$ is the expected annual loss due to element j when not-seismically upgraded and $EAL_{j,SU}$ is the expected annual loss due to element j when seismically upgraded. AM is an amortization conversion expressed by:

$$AM = \left(1 - \frac{1}{(1+r)^t}\right) r^{-1} \tag{3. 6}$$

where r is the internal rate of return or discount rate, and t is the expected occupancy time of the building in years. These parameters are assumed depending on the owner's profile. A comprehensive discussion on different building owner profiles and their influence on building seismic upgrade recommendations is provided in Steneker et al. [2020]. For instance, a long-term building owner with a low-risk investment strategy could be represented by a 4% internal rate of return with a 40-year occupancy time. On the other hand, a 12% internal rate of return and a 10-year occupancy time could be used to represent a high-yield and short-term owner with a higher acceptable risk level.

In order to calculate the expected annual loss due to each non-structural element, hazard curve, collapse fragility curve and average EDPs (i.e. interstorey drift ratio, peak floor acceleration and residual drift) at different earthquake intensity levels are required as input data. For each seismic intensity IM, expected losses due to each non-structural element $E[Lj|IM]$ are calculated as:

$$E\left[L_j \middle| IM\right] = E\left[L_j \middle| NC \cap R, IM\right] \cdot P(NC \cap R | IM) \tag{3. 7}$$

For the seismically upgraded and not-seismically upgraded configuration, Equation 3.7 can be rewritten as:

$$E\left[L_{j,SU} \middle| IM\right] = E\left[L_{j,SU} \middle| NC \cap R, IM\right] \cdot \{1 - P(D|NC, IM)\} \cdot \{1 - P(C|IM)\} \tag{3. 8}$$

$$E\left[L_{j,NSU} \middle| IM\right] = E\left[L_{j,NSU} \middle| NC \cap R, IM\right] \cdot \{1 - P(D|NC, IM)\} \cdot \{1 - P(C|IM)\} \tag{3. 9}$$

$P(C|IM)$ is the probability of collapse of the building given an earthquake with a ground motion intensity IM, which can be obtained from the collapse fragility curve. $P(D|NC,IM)$ is the probability that the building is not repairable given that the structure does not collapse when subjected to a ground motion intensity IM. This probability can be obtained by using the maximum residual drift ratio, together with a building repair fragility, which expresses the probability that the building is not repairable given a residual dift ratio.

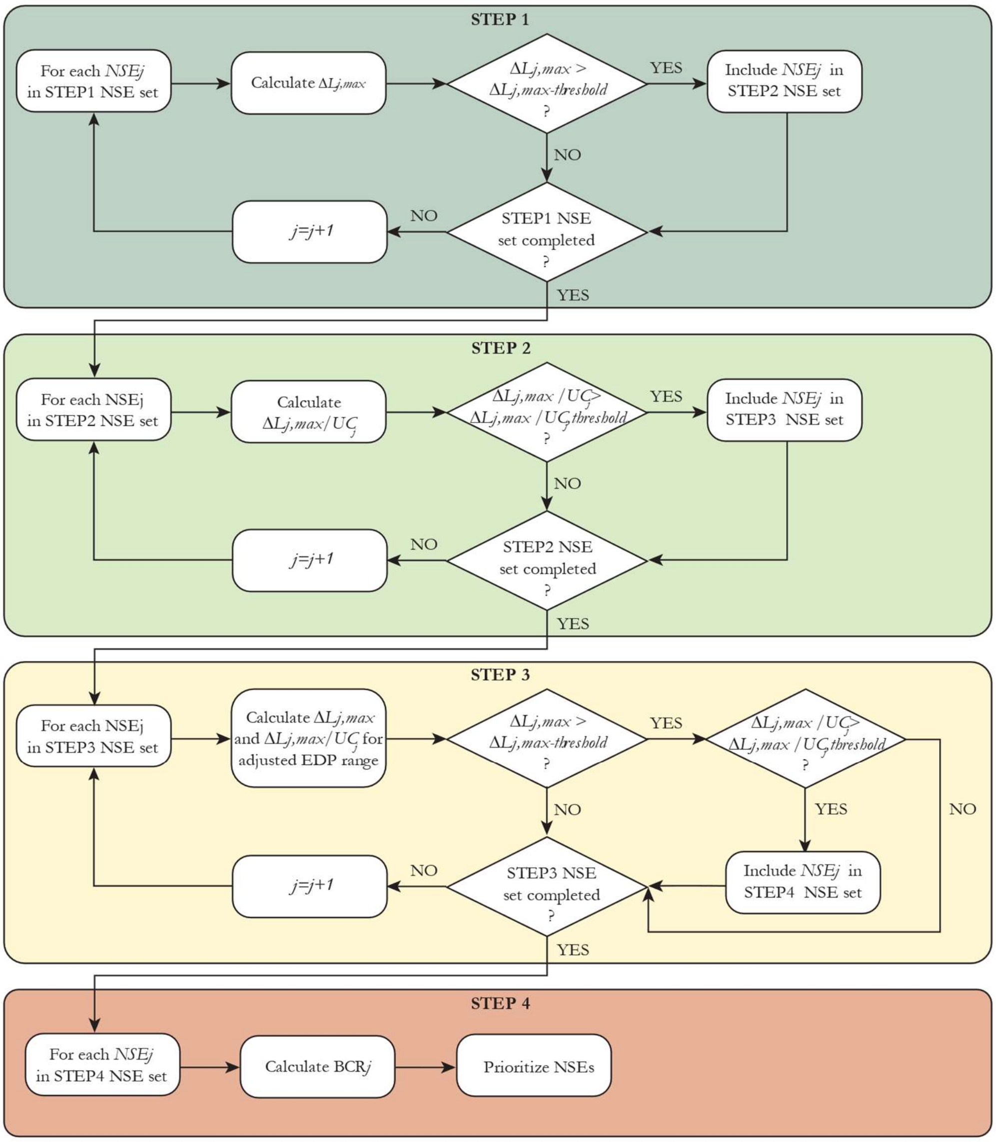

Figure 3.4 Framework Overview.

Following the FEMA P-58 guidelines, a lognormal distribution with a median value of 1% residual drift ratio, and a dispersion of 0.3 can be assumed as repair fragility for a typical building. $E[L_{j,SU}|NC \cap R, IM]$ and $E[L_{j,NSU}|NC \cap R, IM]$ are the expected losses in the seismically upgraded and not seismically upgraded *jth* component, respectively when the building does not collapse, is repaired and is subjected to an earthquake with intensity measure *IM*. These losses can be calculated from the element upgraded and not upgraded Loss-EDP functions using the average EDP values at IM. After the expected element loss at each seismic intensity level has been computed, the expected annual loss corresponding to the not-seismically upgraded and seimcally upgraded configuration ($EAL_{j,SU}$ and $EAL_{j,NSU}$) are obtained by integrated over all considered seismic intensities:

$$EAL_{j,SU} = \int E\left[L_{j,SU}|IM\right]\left|\frac{d\lambda}{dIM}\right| dIM$$

$$EAL_{j,NSU} = \int E\left[L_{j,NSU}|IM\right]\left|\frac{d\lambda}{dIM}\right| dIM$$

3.5 DEVELOPMENT OF NON-STRUCTURAL UPGRADE ASSESSMENT TOOL

To facilitate the practical implementation of the framework, an Excel tool has been developed. In this paragraph a general overview of the tool is provided, while specific instructions for users are provided in Appendix C.

The tool is divided into different Excel sheets that are used to input data, run the framework, perform the calculation, and contain the component database of the FEMA-P58. Users need to interact with the following Excel sheet: “Notes”, “Input”, “Step 0”, “Step 1”, “Step 2”, “Step 3”, “Step4”. The “Notes” sheet provides some bibliographical information on the tool, while the “Input” sheet can be used to input data required to perform each step of the framework. Figure 3.5 shows a screen capture of some portions of the "Input" sheet. The input data for each step are highlighted with a different colour. The FEMA P-58 component database is included in the tool, and elements can be added to the performance model using a dropdown menu. However, users can modify the database and add new components by modifying the Excel sheet “Fragility Database”, “FEMA P-58 PERFORMANCE DATA” and “FEMA P-58 COST DATA”.

Before runing the four steps of the framework, the “Step 0” Excel sheet can be used to estimate element quantities based on the normative quantities provided with FEMA-P58. A separate Excel sheet is dedicated to each step of the framework. Figure 3.6 shows an example of the “Step 1” Excel sheet. Once the step is run, using the “Run Step” button, ΔL_{max} is calculated for each element included in the performance model and compared with the ΔL_{max} threshold specified by the user in the “Input” sheet. Only the element upgrades with ΔL_{max} above the threshold will appear on the “Step 2” Excel sheet. When running Step 2, the $\Delta L_{max}/UC$ ratio is calculated for all the non-structural elements remaining from

Step 1. Only the element with a $\Delta L_{max}/UC$ ratio above the threshold specified in the "Input" sheet will appear on the "Step 3" sheet. The "Run Step" button in the "Step 3" Excel sheet allows to calculate ΔL_{max} and $\Delta L_{max}/UC$ for an EDP range of interest adjusted based on the input maximum expected EDPs. Only components with ΔL_{max} and $\Delta L_{max}/UC$ greater than the specified thresholds will appear on the "Step 4" Excel Sheet. When running the "Run Step" button on the "Step 4" Excel sheet, elements are prioritized according to their benefit-cost ratio.

Input Data - STEP 1
Input Data - STEP 2
Input Data - STEP 3
Input Data - STEP 4

Occupancy	OFFICE
Number of floors	7
Building Cost [$]	12509332.76
ΔLmax Threshold [% Building Cost]	0.10
ΔL max / Upgrade Cost Threshold	0.50

Component Name	ID Before Upgrade	ID After Upgrade
Wall Partitions*	C1011.001a_updated	C1011.001c_updated
Curtain Wall Glazing*	B2022.001_updated	B2022.002_updated
Raised Access Floor	C3027.001	C3027.002
Suspended Ceilings	C3032.001b	C3032.004b
Cold Water Piping, Small Diameter - Piping Fragility	D2021.011a	D2021.014a
Cold Water Piping, Small Diameter - Bracing Fragility	D2021.011b	D2021.014b
Hot Wate Piping, Small Diameter - Piping Fragility	D2022.011a_UpdatedDispersion	D2022.014a_UpdatedDispersion
Hot Wate Piping, Small Diameter - Bracing Fragility	D2022.011b_UpdatedDispersion	D2022.014b_UpdatedDispersion
Hot Wate Piping, Large Diameter - Piping Fragility	D2022.023a_UpdatedDispersion	D2022.024a_UpdatedDispersion
Hot Wate Piping, Large Diameter - Bracing Fragility	D2022.023b_UpdatedDispersion	D2022.024b_UpdatedDispersion
Sanitary Piping - Piping Fragility	D2031.021a_UpdatedDispersion	D2031.024a_UpdatedDispersion
Sanitary Piping - Bracing Fragility	D2031.021b_UpdatedDispersion	D2031.024b_UpdatedDispersion
HVAC duct, Small Area	D3041.011a	D3041.011c
HVAC duct, Large Area	D3041.012a	D3041.012d
HAVC diffuser	D3041.031a	D3041.032d
Sprinkler Piping	D4011.021a	D4011.024a
Sprinkler head	D4011.031a	D4011.053a
Stairs*	C2011.021b_updated	C2011.021a_updated
Elevators	D1014.012	D1014.011
Chiller	D3031.011c	D3031.013h
Cooling Tower	D3031.021c	D3031.023h
Air Handling Unit	D3052.011d	D3052.013k
Motor Control	D5012.013a	D5012.013c
Low Voltage Switchgear	D5012.021b	D5012.023e

Drift max Direction 1 [rad]	0.03
Drift max Direction 2 [rad]	0.03
PFA max Direction 1 [g]	2.32
PFA max Direction 2 [g]	2.32

Collapse Fragility	
Median	1.64
Dispersion	0.55

Demolition Fragility	
Median Irreparable Residual Drift	0.01
Dispersion	0.3

Figure 3.5 Screen capture of some portions of the "Input Data"Excel Sheet of the Non-Structural Upgrade Assessment Tool (see Appendix C).

For each step, a graphical representation of the output is generated by the tool. As the four steps of the framework should be run sequentially, the "Run Step" button on each Excel sheet is enabled only when the previous step has been run. Also, when the "Reset Step" Button of a step is clicked, all the following steps are reset and should be run again.

Component Name	ΔLmax [$]	DLmax [% BC]	DLmax Threshold [% BC]	Pass Step 1 ?	Filter
Wall Partitions*	**260215.59**	**2.08**	**0.50**	**YES**	**1**
Curtain Wall Glazing*	**169764.46**	**1.36**	**0.50**	**YES**	**1**
Raised Access Floor	26257.63	0.21	0.50	NO	1
Suspended Ceilings	**387941.38**	**3.10**	**0.50**	**YES**	**1**
Cold Water Piping, Small Diameter - Piping Fragility	2535.21	0.02	0.50	NO	1
Cold Water Piping, Small Diameter - Bracing Fragility	7813.66	0.06	0.50	NO	1
Hot Wate Piping, Small Diameter - Piping Fragility	0.00	0.00	0.50	NO	1
Hot Wate Piping, Small Diameter - Bracing Fragility	15562.51	0.12	0.50	NO	1
Hot Wate Piping, Large Diameter - Piping Fragility	0.00	0.00	0.50	NO	1
Hot Wate Piping, Large Diameter - Bracing Fragility	0.00	0.00	0.50	NO	1
Sanitary Piping - Piping Fragility	4469.80	0.04	0.50	NO	1
Sanitary Piping - Bracing Fragility	21922.43	0.18	0.50	NO	1
HVAC duct, Small Area	0.00	0.00	0.50	NO	1
HVAC duct, Large Area	5594.12	0.04	0.50	NO	1
HAVC diffuser	16180.80	0.13	0.50	NO	1
Sprinkler Piping	9094.19	0.07	0.50	NO	1
Sprinkler head	1517.92	0.01	0.50	NO	1
Stairs*	**80556.59**	**0.64**	**0.50**	**YES**	**1**
Elevators	20733.82	0.17	0.50	NO	1
Chiller	**237102.33**	**1.90**	**0.50**	**YES**	**1**
Cooling Tower	**119456.78**	**0.95**	**0.50**	**YES**	**1**
Air Handling Unit	**127951.57**	**1.02**	**0.50**	**YES**	**1**
Motor Control	10535.35	0.08	0.50	NO	1
Low Voltage Switchgear	**64737.04**	**0.52**	**0.50**	**YES**	**1**

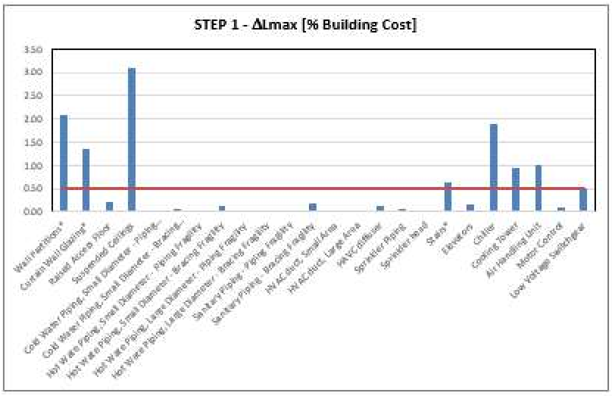

Figure 3.6 Screen capture of "Step 1" Excel Sheet of the Non-Structural Upgrade Assessment Tool (see Appendix C).

3.6 REVIEW AND DISCUSSION

A framework for the assessment of non-structural upgrades has been presented in this chapter. The framework comprises four steps with different amount and sophistication levels of the input data. The steps are intended to be used sequentially. The first three steps are used to remove non-structural upgrades that would not contribute significantly to seismic loss reduction, while the fourth step is used to prioritize non-structural upgrades that remain after Step 3. EDP-Loss functions tailored to the seismic design rating of each element are used in order to assess the benefit of the non-structural upgrades and a benefit-cost ratio is used as a metric to perform the prioritization. An overview of the Non-Structural upgrade assessment Tool to implement the framework has also been provided in this chapter. The results from the application of the framework to three case study buildings will be presented in the next chapter, and compared with the results obtained from the full FEMA P-58 methodology.

4.NON-STRUCTURAL UPGRADE ASSESSMENT FRAMEWORK APPLICATION TO STEEL MOMENT RESISTING FRAMES

4.1 CHAPTER OVERVIEW

In this chapter, the non-structural upgrade assessment framework presented in Chapter 3 is applied to three steel moment resisting frames. Following a description of the three archetype buildings, the results of each step of the framework are presented and discussed. The last part of the chapter is dedicated to the framework's validation, which is performed using the Performance Assessment Calculation Tool (PACT) that implements the FEMA P-58 methodology (FEMA P-58-1, 2018).

4.2 DESCRIPTION OF THE ARCHETYPE BUILDINGS

The three archetype buildings used in this chapter were selected from the SAC steel project [ATC, 1994]. The buildings are assumed to be located in the city of Los Angeles, United States, and are designed according to the 1994 Uniform Building Code [ICBO, 1994].

In the three investigated archetype buildings, the seismic force-resisting system is composed of moment-resisting frames along the building's perimeter, while interior frames are designed to carry only gravity loads. The three-storey archetype building was adapted from the FEMA 440 document [FEMA, 2005]. It comprises three bays in the North-South direction and six bays in the East-West direction, with a total floor area of 2007 m^2. The six-storey building, originally studied by Tsai and Popov [1988] and modified by Hall [1995], is composed of three bays in the North-South direction and braced frames with four bays in the East-West direction with a total floor area of 803 m^2. The nine-storey archetype building, also adapted from the FEMA 440 document [FEMA 440, 2005], comprises five bays in both directions and has a floor area of 2090 m^2.

For this study, only the North-South direction of the three archetype buildings was considered (Figure 4.1). In accordance with Chalarca et al. [2020], the modeling of the three frames was implemented in the OpenSees software [McKenna *et al.*, 2010] using *BeamWithHinges* elements and *Steel02* material to model beams and columns of each moment-resisting frame. The interior gravity frames were modeled using a leaning gravity column to account for P-Delta effects. The computed fundamental periods of the three archetype buildings are summarized in Table 4.1.

Table 4.1. Archetype building fundamental periods

	three-storey building	six-storey building	nine-storey building
T_1 [s]	0.79	1.48	1.89

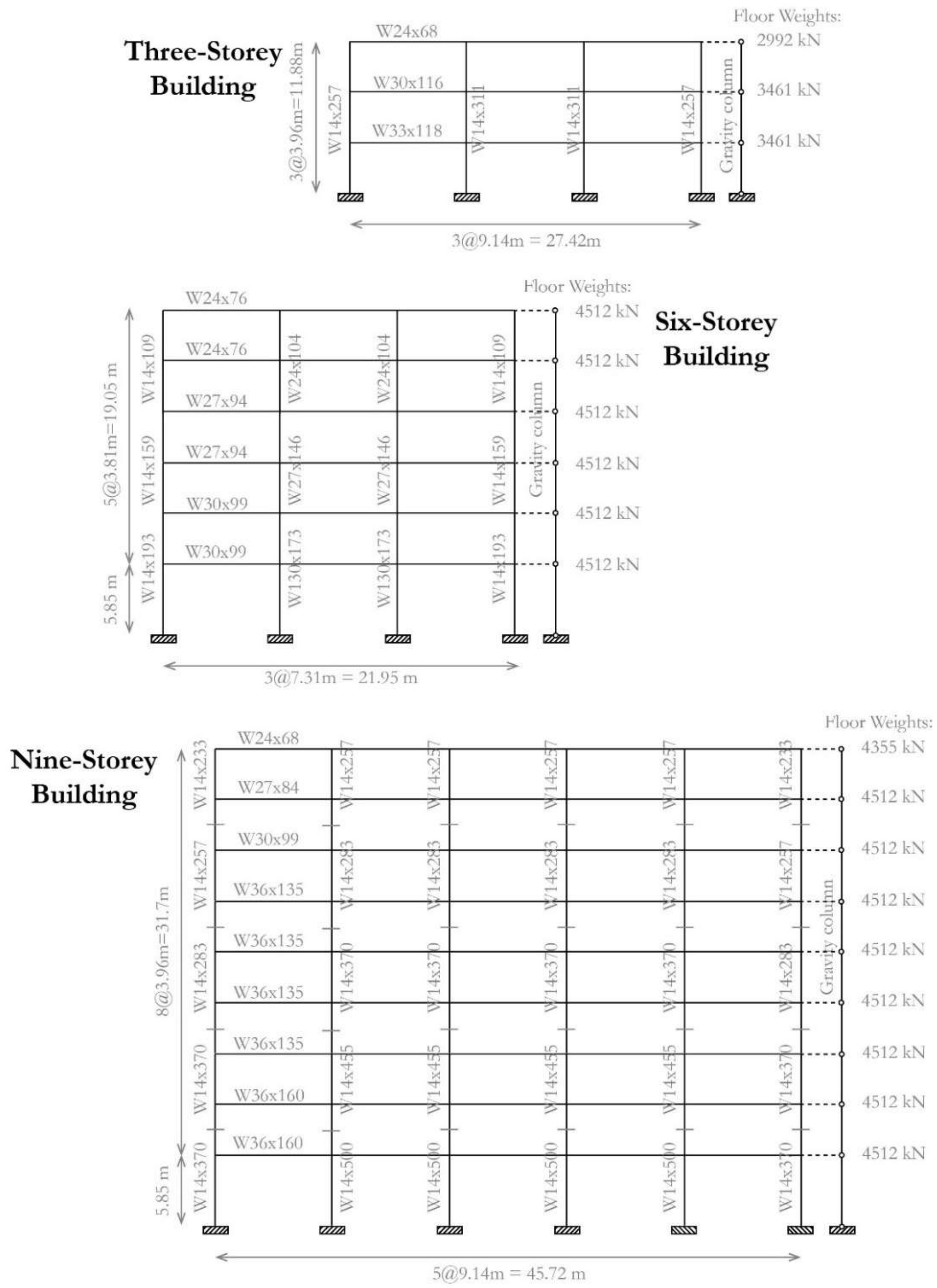

Figure 4.1 Steel moment resisting frame archetype buildings.

4.3 NON-STRUCTURAL UPGRADE ASSESSMENT FRAMEWORK APPLICATION

4.3.1 Step 1

The first step of the non-structural upgrade assessment framework requires the input data listed in Table 4.2 and Table 4.3. Table 4.2 summarizes the following data: building occupancy, number of storeys, building cost and floor area. In Table 4.3, the non-structural element upgrades included in the building performance models are presented. NSE upgrades used in this chapter were taken from Steneker et al. [2020], and the FEMA P-58 fragility and consequence functions were used. The fragility functions of the non-structural elements in the seismically upgraded and not-seismically upgraded configuration are presented in Appendix B. Non-structural element quantities are summarized in Table 4.4. The quantity estimation was performed using the Quantity Estimation Tool provided with the FEMA P-58 methodology (FEMA P-58-3, 2018). Although no structural analyses are required to run Step 1, an EDP range of engineering interest should be assumed. In this initial phase of the framework, an EDP range with a maximum value of Peak Interstorey Drift (PID) of 5% and a maximum value of Peak Floor Accelerations (PFA) of 5 g was assumed. This value of PFA is quite high and it is not likely to be achieved by the investigated archetype buildings. For illustration purposes, it was assumed, nevertheless, with the aim of highlighting the differences between a very large EDP range assumed in the first step and the reduced EDP range assumed in the third step of the framework, when more information on the structural response is available. Depending on their intent and available resources, users of the framework might want to make different considerations for the EDP range to assume in this first step.

Table 4.2. Step 1 input data: general information on the archetype buildings

	three-storey Building	**six-storey Building**	**nine-storey Building**
Occupancy	Office	Office	Office
Number of Storey	3	6	9
Building Cost [$]	15000000	12509333	43000000
Floor Area per level [SF]	21600	8640	22500

Table 4.3. Step 1 input data: non-structural upgrades implemented in the archetype buildings

Component Name	**Component ID Before Upgrade**	**Component ID After Upgrade**
Wall Partitions*	C1011.001a	C1011.001c
Curtain Wall Glazing*	B2022.001	B2022.002
Raised Access Floor	C3027.001	C3027.002

Suspended Ceilings	C3032.001b	C3032.004b
Cold Water Piping, Small Diameter	D2021.011a - D2021.011b	D2021.014a - D2021.014b
Hot Wate Piping, Small Diameter	D2022.011a - D2022.011b	D2022.014a - D2022.014b
Hot Wate Piping, Large Diameter	D2022.023a - D2022.023b	D2022.024a - D2022.024b
Sanitary Piping	D2031.021a - D2031.021b	D2031.024a - D2031.024b
HVAC duct, Small Area	D3041.011a	D3041.011c
HVAC duct, Large Area	D3041.012a	D3041.012d
HAVC diffuser	D3041.031a	D3041.032d
Sprinkler Piping	D4011.021a	D4011.024a
Sprinkler head	D4011.031a	D4011.053a
Stairs*	C2011.021b	C2011.021a
Elevators	D1014.012	D1014.011
Chiller	D3031.011c	D3031.013h
Cooling Tower	D3031.021c	D3031.023h
Air Handling Unit	D3052.011d	D3052.013k
Motor Control	D5012.013a	D5012.013c
Low Voltage Switchgear	D5012.021b	D5012.023e

Source: Data from FEMA P-58-2 [2018].

* Component fragility curves were updated based on Steneker et al.[2020].

Table 4.4. Non-structural element quantities

Component Name	Component Quantity						Unit *
	three-storey Building		six-storey Building		nine-storey Building		
	Floors 1-3	Roof	Floors 1-6	Roof	Floors 1 -9	Roof	
Wall Partitions	21.6	0.0	8.6	0.0	22.5	0.0	LF 100
Curtain Wall Glazing	216.0	0.0	86.4	0.0	225.0	0.0	SF 30
Raised Access Floor	162.0	0.0	64.8	0.0	168.8	0.0	SF 100
Suspended Ceilings	36.0	0.0	14.4	0.0	37.5	0.0	SF 600
Cold Water Piping, Small Dia.	0.9	0.0	0.4	0.0	0.9	0.0	LF 1000

Hot Water Piping, Small Dia.	1.8	0.0	0.7	0.0	1.9	0.0	LF 1000
Hot Water Piping, Large Dia.	0.6	0.0	0.3	0.0	0.7	0.0	LF 1000
Sanitary Piping	1.2	0.0	0.5	0.0	1.3	0.0	LF 1000
HVAC duct, Small Area	1.6	0.0	0.6	0.0	1.7	0.0	LF 1000
HVAC duct, Large Area	0.4	0.0	0.2	0.0	0.5	0.0	LF 1000
HAVC diffuser	19.4	0.0	7.8	0.0	20.3	0.0	EA 10
Sprinkler Piping	4.3	0.0	1.7	0.0	4.5	0.0	LF 1000
Sprinkler head	1.9	0.0	0.8	0.0	2.0	0.0	EA 100
Stairs	3.0	0.0	1.0	0.0	3.0	0.0	EA 1
Elevators**	2.0	0.0	2.0	0.0	6.0	0.0	EA 1
Chiller	0.0	1.0	0.0	1.0	0.0	2.0	TN 500
Cooling Tower	0.0	1.0	0.0	1.0	0.0	2.0	TN 500
Air Handling Unit	0.0	2.0	0.0	2.0	0.0	5.0	CF 30000
Motor Control	0.0	3.0	0.0	3.0	0.0	9.0	EA 1
Low Voltage Switchgear	1.0	0.0	1.0	0.0	1.0	0.0	AP 400

*LF = linear feet = 0.3048 meter; SF = square feet = 0.0929 square meter; EA = per unit; TN = ton; CF = Cubic feet / minute = 0.0283 cubic meter / minute;

AP = Amp.

** Elevators are assumed to be located at the ground floor (Floor 1) to perform the analyses

To run Step 1, following the procedure described in Chapter 4, Loss-EDP functions of each component in the seismically upgraded and original (not-seismically upgraded) configurations were calculated. The maximum loss reduction ΔL_{max} of each upgrade in the assumed EDP range of interest was used as a parameter to assess the non-structural upgrades. A $\Delta L_{max,threshold}$ equal to 0.1% of the building cost was assumed for illustration purposes, and NSE upgrades with ΔL_{max} lower than 0.1% of the building cost were removed from the non-structural element set to be assessed in Step 2.

Figure 4.2 shows the results of Step 1. Similar results were obtained for the three archetype buildings. Pipings, HVAC ducts and motor control centers have the smallest values of ΔL_{max}, which are below the assumed threshold. Therefore, these components are removed from the NSE set to be assessed in Step 2. Suspended ceilings are found to have the largest values of ΔL_{max}, followed by partition walls, NSEs located at the roof level such as chillers, cooling towers and air handling units, and drift sensitive NSEs such as curtain walls and stairs.

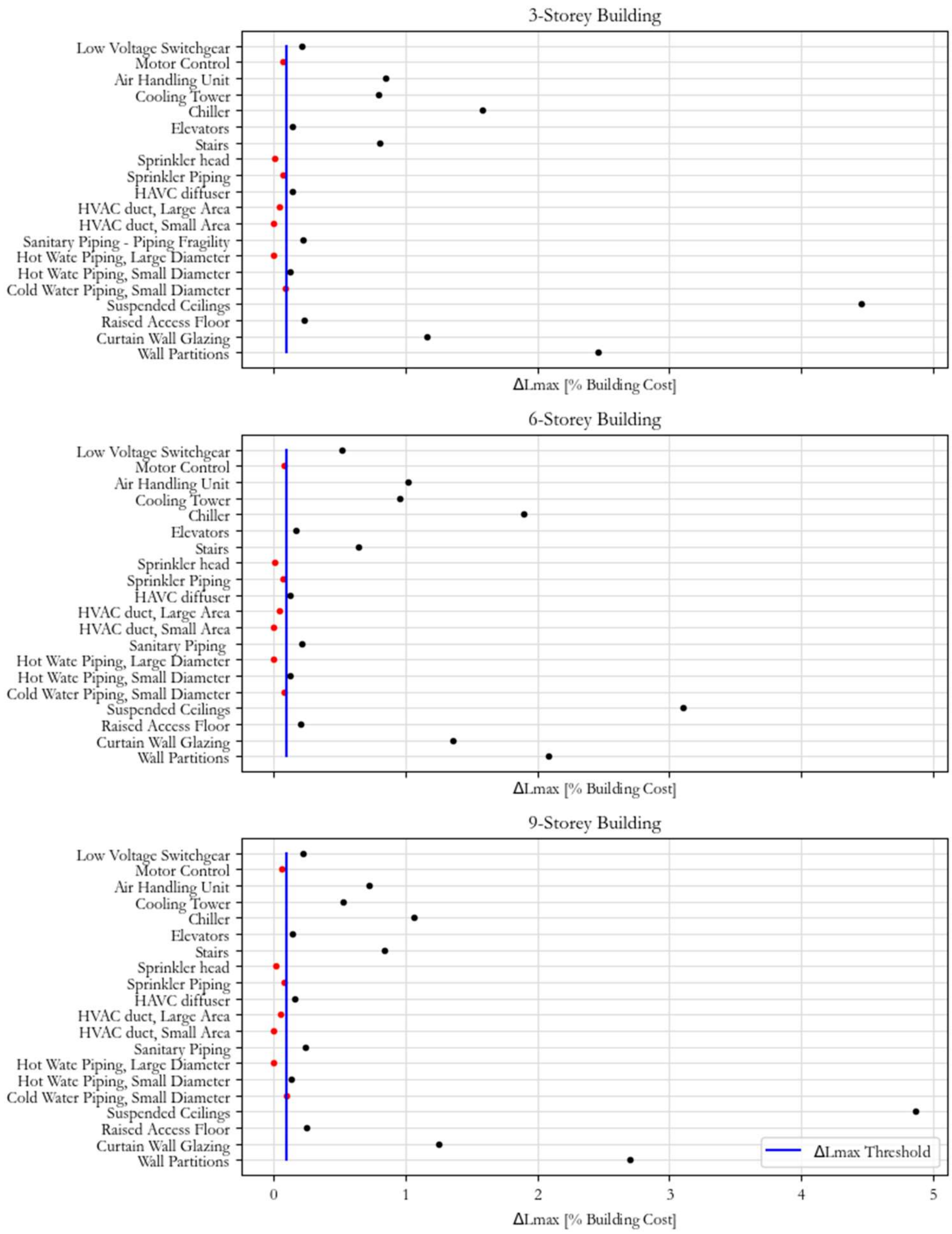

Figure 4.2 Step 1 results for the three archetype buildings.

As discussed in Chapter 3, ΔL_{max} measures the potential maximum loss reduction within an assumed EDP range and cannot be used to prioritize NSEs. This is because, depending on the structural response, ΔL_{max} values in Figure 4.2 may not be reached. For example, Figure 4.3 shows the Loss-EDP functions and the ΔL_{max} values of sprinkler pipings, chiller and suspending ceilings in the three-storey archetype building.

Sprinkler pipings have a ΔL_{max} value equal to 0.08% of the building cost, which is below the specified threshold. This component is thus removed from the NSE set to be assessed in step 2 because, independently from the structural response, the expected loss reduction due to the upgrade of sprinkler piping would be below 0.1% of the building cost. From the Loss-EDP functions of the chiller and suspended ceilings, it can be noticed that ΔL_{max} is above the threshold for both components but it occurs at different EDP values. The chiller contributes to a large loss reduction at PFA values lower than 0.5 g, while the maximum loss reduction in suspended ceilings occurs at a PFA value of about 2 g. Both NSEs can potentially contribute to a significant loss reduction when seismically upgraded. However, at this first step of the framework, when no information on the structural response is available, it is not straightforward to identify which NSE upgrades will contribute more to the building seismic loss reduction. For this reason, both NSE upgrades will be further investigated in the next steps.

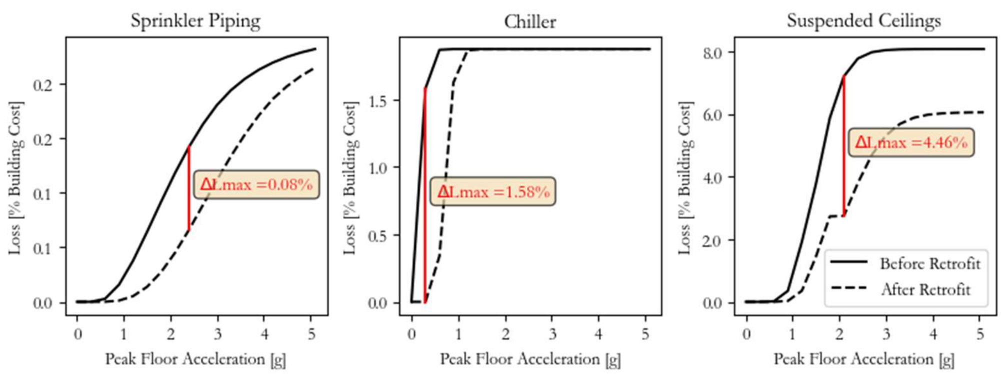

Figure 4.3 Example of NSE Loss-EDP functions for the three-storey archetype building.

4.3.2 Step 2

Non-structural upgrade costs to run Step 2 were taken from the literature (Steneker et al., 2020). Upgrade costs are calculated assuming that non-structural elements are retrofitted, which includes all tasks necessary to remove and replace NSEs. The ratio $\Delta L_{max}/UC$ between the ΔL_{max} parameter calculated in Step 1 and the upgrade cost UC is used as a parameter to eliminate non-structural element upgrades in Step 2. A $\Delta L_{max}/UC$ ratio equal to 0.5 was assumed for illustration as a threshold.

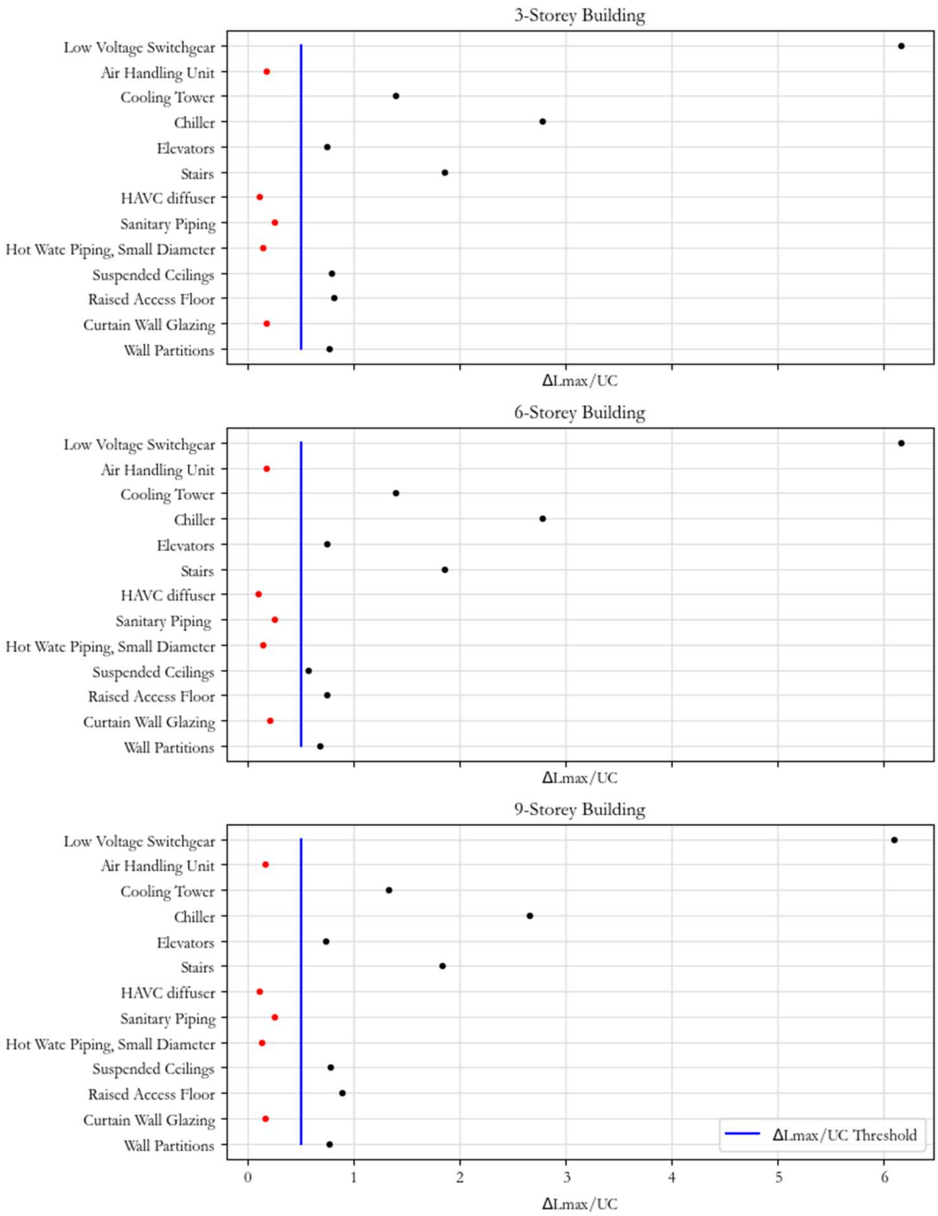

Figure 4.4 Step 2 results for the three archetype buildings.

Figure 4.4 shows the $\Delta L_{max}/UC$ ratio calculated for all non-structural upgrades that have remained after Step 1. Some non-structural upgrades with a low value of ΔL_{max} in Step 1 also have a low value of $\Delta L_{max}/UC$ ratio. For example, this is the case of sanitary pipings and HVAC diffusers that have a $\Delta L_{max}/UC$ ratio below the assumed threshold and are therefore removed from the non-structural upgrades to be assessed in Step 3. The upgrade of curtain walls is also removed from the NSE upgrade set to be assessed in Step 3. For this component, a large value of ΔL_{max} was obtained in Step 1 but values of $\Delta L_{max}/UC$ below 0.5 were obtained in Step 2. This indicates that, although the upgrade of curtain walls could potentially contribute to provide a significant seismic loss reduction, the cost of the upgrade is so high compared to its potential benfit that the upgrade is considered not viable and is not investigated further.

4.3.3 Step 3

Step 3 needs as additional input the maximum expected values of Peak Interstorey Drift (PID) and Peak Floor Acceleration (PFA) in the investigated buildings. As discussed in Chapter 3, depending on the analyst's intent/resources, time history or more simplified analyses can be used to obtain Step 3 input data. For the archetype building examples discussed in this chapter, time history analyses were used to obtain the maximum PID and PFA values at the Maximum Considered Earthquake level MCE (2% probability of exceedance in 50 years) (Table 4.5).

Table 4.5. Input data Step 3: Maximum PID and PFA at MCE

	three-storey building	six-storey building	nine-storey building
Maximum PID at MCE	3.11%	3.09%	3.04%
Maximum PFA at MCE [g]	1.81	2.32	2.41

The 44 ground motion records from the FEMA P-695 far-field set (FEMA P695, 2009) were used to run time history analyses. The site response spectrum at MCE was obtained using the USGS (United States Geological Survey) Uniform Hazard Tool [USGS, 2014] and, following the ASCE 7-22 (ASCE, 2022) procedure, the records were scaled such that their average 5% damped response spectrum does not fall below 90% of the target response spectrum in a specified period range. According to the ASCE procedure, the upper bound of the period range must be greater than or equal to twice the fundamental period of the building T_1, while the lower bound must include at least the number of elastic modes necessary to achieve 90% mass participation and the lower bound period must not exceed $0.2T_1$. Figure 4.5 shows the scaling of the records for the six-storey building, considering as target response spectrum the site response spectrum at the MCE level.

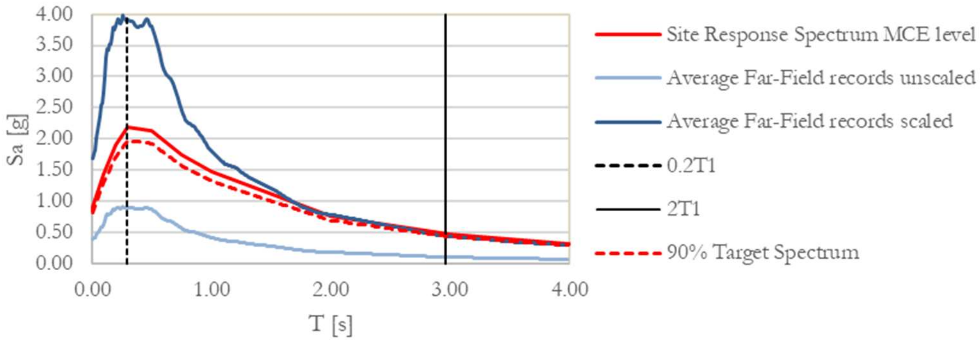

Figure 4.5 Example of scaling procedure for the six-storey building.

The PID and PFA values in Table 4.5 were used to define a new EDP range, and the ΔL_{max} and ΔL_{max} $/UC$ ratio parameters corresponding to this new EDP range were calculated. The results of the third step of the framework are illustrated in Figure 4.5.

As the assumed upgrade costs are the same in all steps of the framework, the difference between the results of Step 3 and the ones of Step 1 and 2 can be attributed to a different value of ΔL_{max} corresponding to the new EDP range. For most non-structural upgrades, the ΔL_{max} and $\Delta L_{max}/UC$ ratio values remain the same as in Step 1 and Step 2. This indicates that the ΔL_{max} value determined in Step 1 occurs at an EDP value that is still inside the new EDP range assumed in Step 3. For example, Figure 4.3 shows that for the three-storey archetype building, the EDP that maximizes the loss reduction due to the upgrade of the chiller is below 1.8 g, which is the maximum EDP value of the new EDP range considered in Step 3. Therefore, the same values of ΔL_{max} and $\Delta L_{max}/UC$ due to the upgrade of the chiller are obtained in Step 3. On the other hand, for suspended ceilings, Figure 4.3 shows that ΔL_{max} in Step 1 occurs at a PFA value above 2 g, which is not included in the new EDP range assumed in Step 3. For this reason, smaller values of ΔL_{max} and $\Delta L_{max}/UC$ are obtained for suspending ceilings in Step 3 compared with Steps 1 and 2. Although for some non-structural upgrades the ΔL_{max} and $\Delta L_{max}/UC$ parameters calculated in Step 3 are lower than the ones calculated in Step 1 and 2, they are still above the assumed thresholds, which are 0.1% of the building cost and 0.5 for ΔL_{max} and $\Delta L_{max}/UC$, respectively. Therefore, all non-structural upgrades assessed in Step 3 are included in the non-structural upgrades set to be investigated in Step 4.

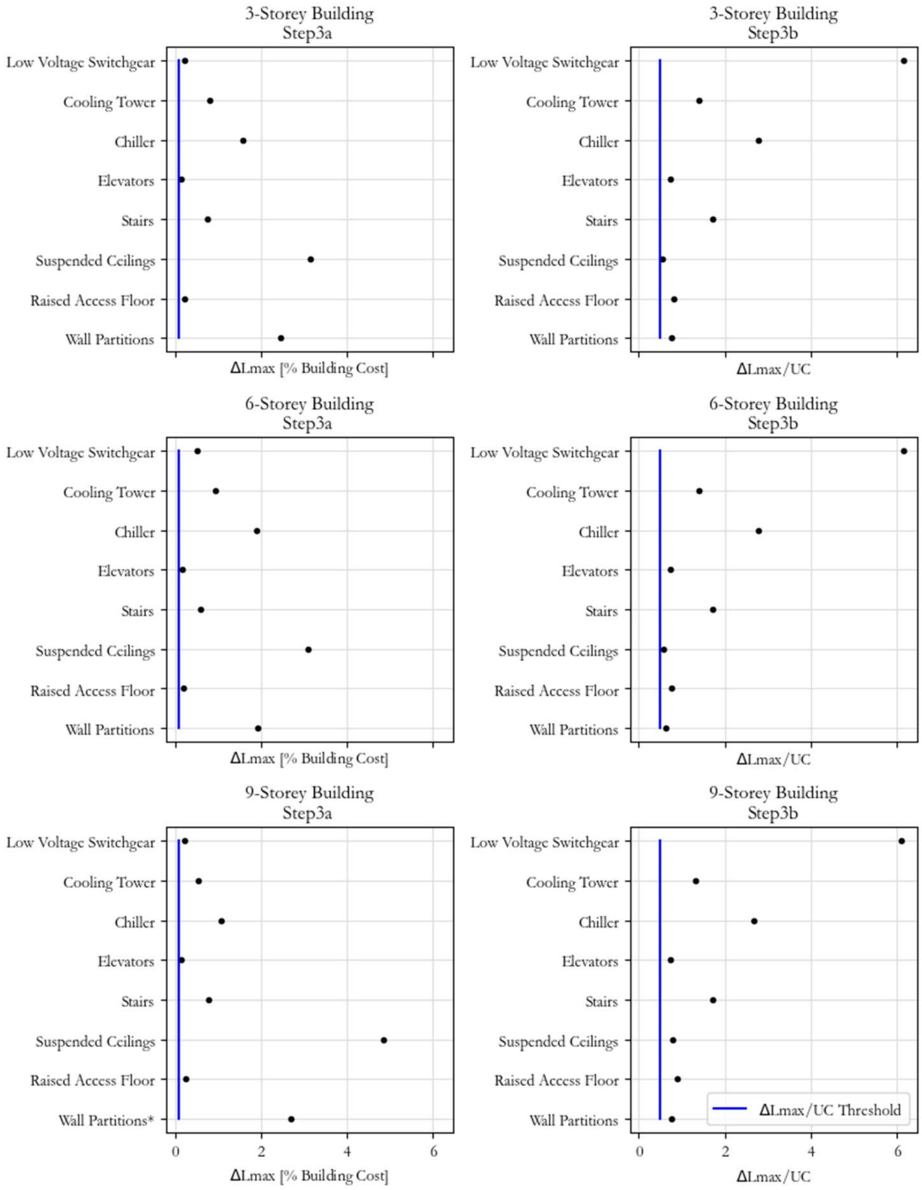

Figure 4.6 Results Step 3 for all archetype buildings

4.3.4 Step 4

The additional input data required to run Step 4 are the hazard curve at the site, structural response parameters for different earthquake intensities (average EDP values and collapse fragility), and parameters to perform the benefit-cost analysis (i.e. expected occupancy time of the building, internal rate of return). The hazard curve at the site was obtained using the USGS Uniform Hazard Tool [USGS, 2014], and the six intensities in Figure 4.7 were considered to run time history analyses.

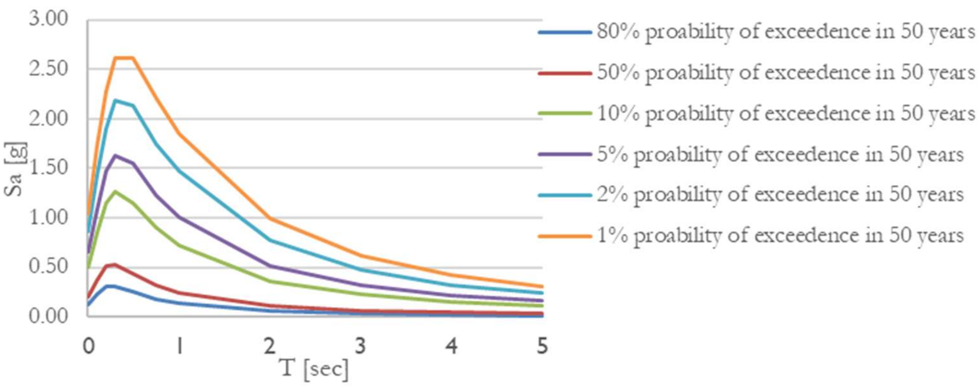

Figure 4.7 Uniform Response Spectra for different seismic intensity levels.

The results of Step 4 are presented in Figure 4.8. The same procedure described in Step 3 was used to scale the 44 records of the FEMA P-695 far-field set at different intensity levels. Time history analyses were used to obtain the average EDP values at different intensity levels and the collapse fragility curve. An expected occupancy time of 40 years and an internal rate of return of 4% were assumed.

Using the procedure described in Chapter 3, the benefit-cost ratio of NSE upgrades that remained after Step 3 was calculated. As illustrated in Figure 4.8, for all archetype buildings, the upgrade of the chillers has the largest benefit-cost ratio, followed by cooling tower and low voltage switchgear. Based on Equation 3.5, an upgrade can be considered viable if the benefit-cost ratio is greater thanone , which indicates that the loss reduction produced by the upgrade exceeds the upgrade implementation cost. For the three-storey building, all non-structural upgrades have a benefit-cost ratio lower than 1; for the six-storey and nine-storey building, only the upgrade of chiller has a benefit-cost ratio larger than 1.

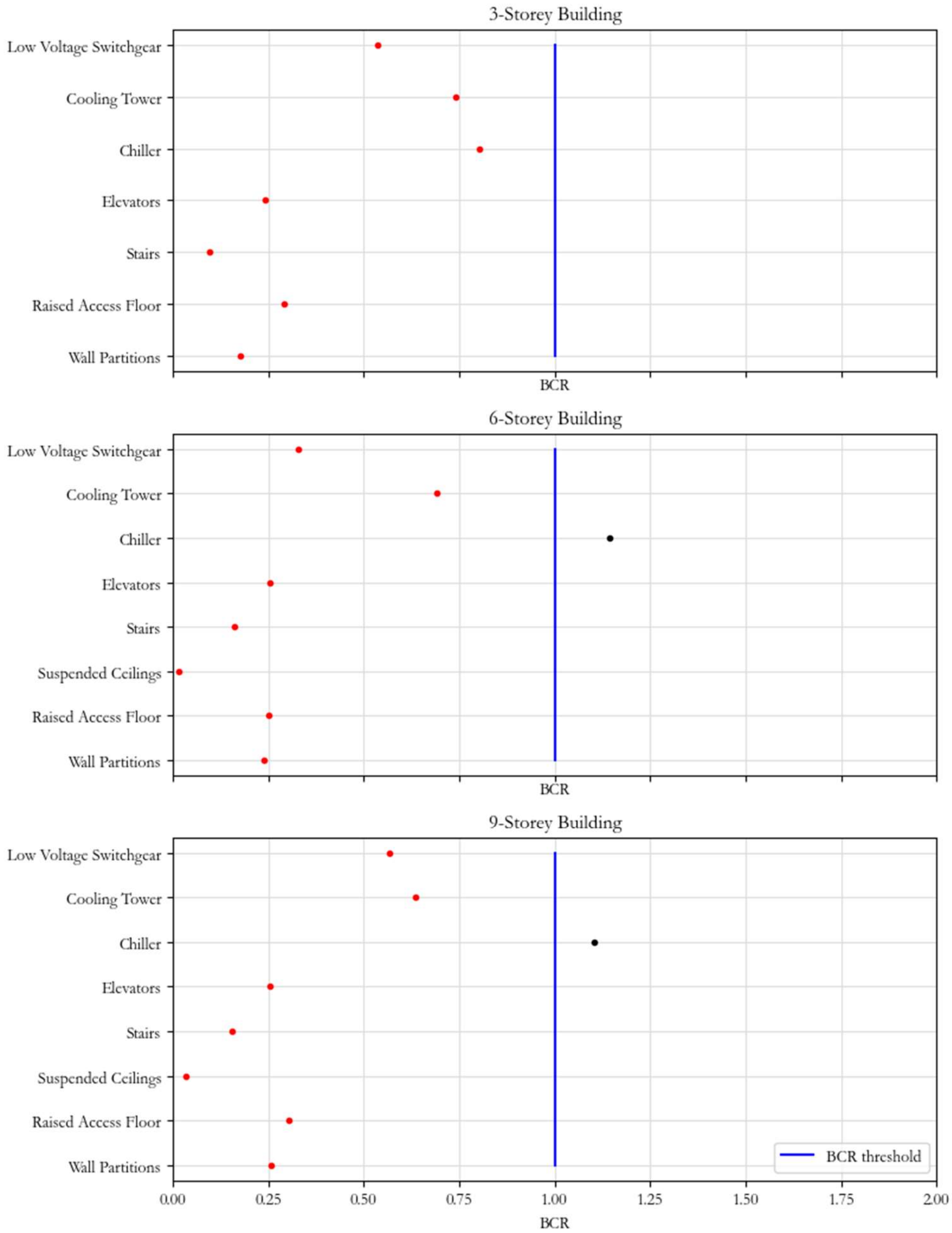

Figure 4.8 Results Step 4 for the three archetype buildings.

4.4 NON-STRUCTURAL UPGRADE ASSESSMENT FRAMEWORK VALIDATION

To validate the results obtained with the proposed framework for the three archetype buildings considered in this chapter, the benefit-cost ratio values obtained using the non-structural upgrade assessment framework were compared with the ones obtained using the FEMA P-58 methodology. The comparison was performed considering all non-structural upgrades included in the performance models and not only the upgrades assessed in Step 4. The first step to obtain the FEMA P-58 benefit-cost ratio values was to implement the building performance models of the three archetype buildings in PACT. The structural response input data used in Step 4 were input in PACT and the expected annual loss corresponding to the building models with all NSEs not seismically upgraded was first calculated. For each non-structural upgrade, the PACT performance model was modified and the expected annual loss of the building with the upgraded component was calculated. The expected annual loss variation due to the upgrade of each component was recalculated and input in Equation 3.5 to obtain the FEMA P-58 benefit-cost ratio values of each non-structural upgrade.

Figure 4.9 shows the comparison between the results obtained using the non-structural upgrade assessment framework and PACT. For NSE upgrades with small BCR values, small differences can be noticed between the BCR values estimated using the proposed framework and PACT. For the three-storey building, the largest differences are obtained for the upgrade of the motor control center. For the six-storey and nine-storey building, the largest discrepancies between the non-structural upgrade assessment framework and PACT are obtained for the chiller. The main reason for these discrepancies is that the non-structural upgrade assessment framework neglects the uncertainty in the structural response and in the repair cost, which are included in the full probabilistic approach used in FEMA P-58 methodology and implemented in PACT through a Monte Carlo simulation. Depending on the Loss-EDP function shape, on the dispersion in the structural response and in the component repair cost, the BCR of a component may be more or less sensitive to the fact that those uncertainties are neglected.

With regard to the prioritization of the non-structural upgrades, the proposed framework is able to identify the most crucial non-structural upgrades that should be implemented in order to maximize the benefit cost-ratio of the investment. For the three-storey building, the FEMA P-58 and the proposed framework both list the chiller, the motor control center, the cooling tower, and the low voltage switchgear as the upgrades that have the highest benefit-to-cost ratio. However, the proposed framework overestimates the benefit-cost ratio of the motor control center upgrade, which results in a slightly different order of upgrades when compared to the FEMA P-58. According to the non-structural upgrade assessment framework, the upgrades would be implemented in the following order: the motor control center, the chiller, the cooling tower, and the low voltage switchgear. Conversely, the order would be the chiller, the motor control center, the low voltage switchgear, and the cooling tower using the FEMA P-58 methodology. For the six-storey building and nine-storey building, the most critical upgrades would be implemented in the following order according to both the proposed framework and the FEMA P-58 methodology: the chiller, the motor control center and the cooling tower.

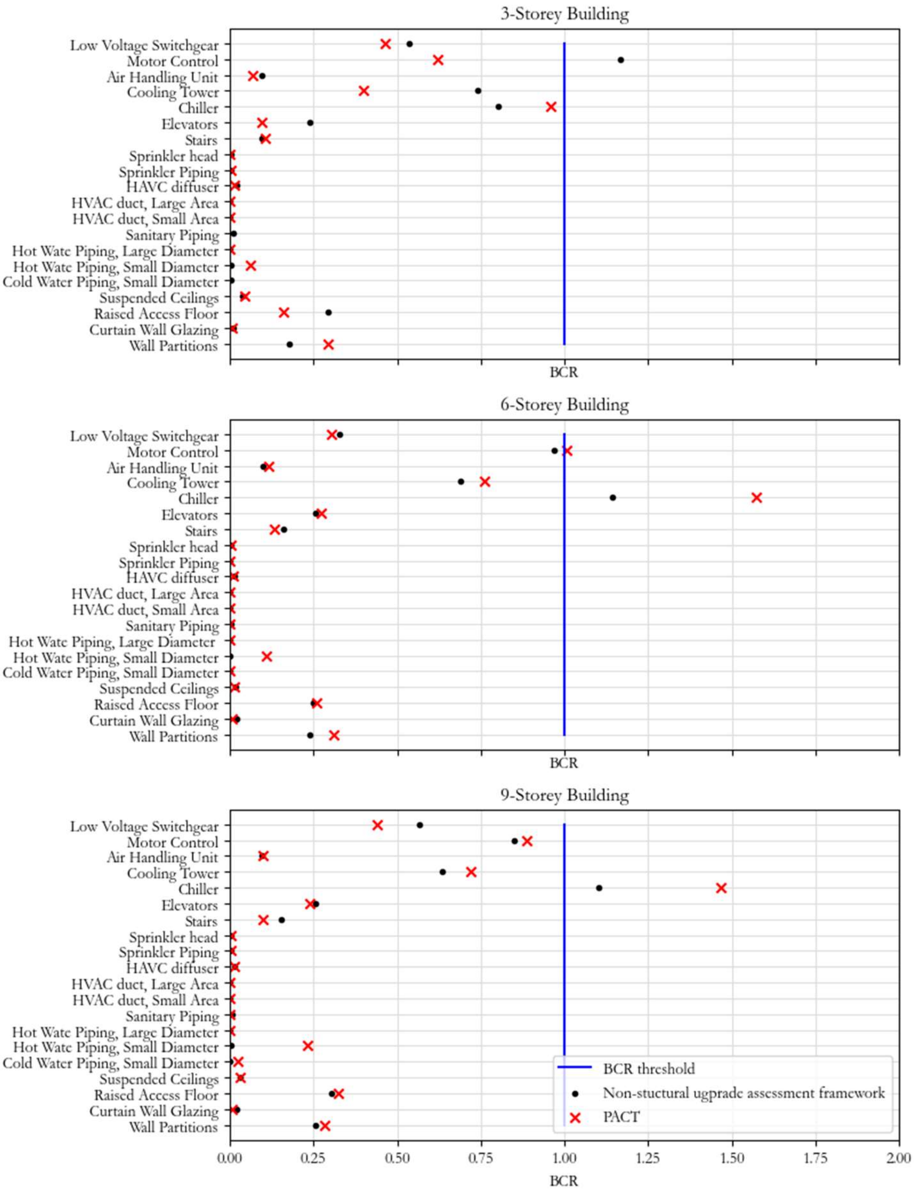

Figure 4.9 Validation of the framework using PACT.

Another aspect to verify using the results in Figure 4.9 is the benefit-cost ratio of non-structural element upgrades removed in the first steps of the framework. From Figure 4.9 it can be noticed that, with the exception of the motor control, all NSE upgrades removed in the first steps of the framework have a low benefit-cost ratio. Among the non-structural upgrades with the highest benefit-cost ratio, the upgrade of the motor control center is the only one that was eliminated in the framework's initial steps. The upgrade of the motor control center was removed in Step 1 (Figure 4.2) because its ΔL_{max} was below 0.1% of the building cost. Therefore, the fact that the motor control center has a high benefit cost-ratio indicates that, even if its upgrade would provide a small contribution to seismic loss reduction, the upgrade cost of the component is so small that a high upgrade benefit-cost ratio would be obtained if the upgrade of the motor control center was included in the non-structural element set assessed in step 4. The assumed thresholds for ΔL_{max} and $\Delta L_{max}/UC$ determine whether an upgrade is removed in the framework's initial steps. Depending on their objectives, analysts should choose which thresholds to assume; additionally, it could be helpful for them to verify the impact of changing the threshold values on the framework outcomes.

4.5 Review and Discussion

The application of the non-structural upgrade assessment framework to three steel moment resisting frame archetype buildings with three, six and nine storeys was discussed in this chapter. For all the archetype buildings, the upgrade of the chiller was found to have the largest benefit-cost ratio, followed by the cooling tower and the low voltage switchgear. For the three-storey building, all non-structural upgrades have a benefit-cost ratio lower than 1; for the six-strorey and nine-strorey building, the upgrade of the chiller has a benefit-cost ratio larger than 1.

The FEMA P-58 methodology was used to validate the framework's results. The benefit-cost ratio values obtained using the proposed framework were compared with the ones obtained using the FEMA P-58 methodology, and it was found that the proposed framework is able to identify the most crucial non-structural upgrades that should be implemented in order to maximize the benefit cost-ratio of the investment. Although the proposed framework is simplified and neglects some sources of uncertainties in the loss estimation process, the results from this chapter show its potential as a simplified tool for assessing NSEs and communicating the benefit of upgrading them to stakeholders at an early stage of the design process.

Compared to using the complete FEMA P-58 methodology to assess non-structural upgrades, the proposed framework presents advantages that make it a practical and efficient tool for the preliminary non-structural element upgrade assessment. A first difference between the FEMA P-58 methodology and the proposed approach is the level of sophistication of the required input data. In order to apply the FEMA P-58 methodology, a model of the structure must be implemented and time history or simplified analyses performed. The proposed framework, on the other hand, is divided into sequential steps that require different levels of sophistication of the input data. In the first step of the framework, users can obtain useful insights on the potential impact of different non-structural upgrades without running any structural analyses, which could be very valuable at the beginning of the decision-making process when limited resources are available. For the archetype buildings investigated in this chapter, for example, users of the framework may immediately recognize from the first step that upgrades to components such as pipings and HVAC ducts would not contribute much to seismic loss reduction.

Another advantage of using the proposed framework is the transparency of the non-structural upgrade assessment process. When running the FEMA P-58 methodology, it is not straightforward to understand why an upgrade is more important than another and how much the non-structural upgrade impact would change if the structural response changes. In the proposed framework, users are guided step by step in identifying which non-structural upgrades may be more impactful. The use of component Loss-EDP functions, which directly relate expected loss with EDP values, improves the transparency of the non-structural upgrade assessment process as it helps understand how much the impact of an upgrade might change depending on the structural response. For instance, in this chapter, the Loss-EDP functions of the sprinkler piping, chiller and the suspending ceilings in the three-storey building were plotted. Just from examining these loss-EDP functions, it is possible to determine that the sprinkler piping would not provide a significant contribution to seismic loss reduction in the EDP range of interest; the chiller could provide a significant contribution for PFA values lower than 0.5 g; and the suspending ceilings provide their maximum contribution to seismic loss reduction for values of PFA above 2 g. Moreover, the FEMA P-58 methodology has been developed to assess the seismic performance of buildings and not to assess non-structural upgrade impact. It can be used for this second purpose but this requires an extensive computational effort as, for each considered non-structural upgrade, the loss estimation must be repeated using a Monte Carlo simulation. On the other hand, the main objective of the proposed framework is not to estimate building seismic loss but to identify viable non-structural upgrades. The framework can be easily implemented on an Excel spreadsheet and the identification of viable non-structural upgrades is straightforward and can be done with a low computational effort.

5.USHOVER-BASED FRAMEWORK TO ASSESS INTEGRATED STRUCTURAL AND NON-STRUCTURAL UPGRADE STRATEGIES

5.1 CHAPTER OVERVIEW

In this chapter, a pushover-based framework to assess integrated structural and non-structural upgrade strategies is proposed. The framework is intended as a preliminary simplified method for evaluating the impact of various integrated structural and non-structural upgrade strategies in a way that is straightforward and consistent with the limited resources available at the early stage of the decision-making process. An overview of the framework and a discussion of the framework's steps are provided in this chapter. The application of the framework to a six-storey moment resisting frame is presented in Chapter 6.

5.2 OVERVIEW OF THE PROPOSED FRAMEWORK

The key steps of the proposed framework are illustrated in Figure 5.1. First, two performance objectives are defined in terms of probability of collapse of the building at the Maximum Credible Earthquake level MCE (2% probability of exceedance in 50 years) and Expected Annual Loss (EAL). These performance objectives are used in the framework to establish if a combination of structural and non-structural upgrades is viable. A viable building seismic upgrade is characterized by a probability of collapse at MCE lower than the probability of collapse performance objective and an EAL lower than the EAL performance objective. After setting the desired performance objectives, the structural seismic design/retrofit alternatives to be investigated should be identified. For instance, a first alternative could be the building in its original (as-built) configuration, without any structural upgrades. Other alternatives could be then defined by considering structural upgrade strategies, such as increase in strength, stiffness and/or damping.

In order to identify which combination of structural and non-structural upgrade strategies are viable, the steps described below and summarized in Figure 5.1 should be followed.

Step 1 - Characterize hazard. The first step is the characterization of the seismic hazard at the site by the definition of the site hazard curve and the seismic response spectra at different seismic intensity levels.

Step 2 - Characterize structural response. For each considered structural design/retrofit alternative, an equivalent Single Degree of Freedom (SDOF) approximation is used to perform the structural analysis. The structural response is defined in terms of collapse fragility curve and Engineering Demand Parameters EDPs (peak interstorey drift, peak floor acceleration and residual drift). The probability of building collapse at MCE is calculated in this step and compared with the probability of collapse performance objective. If the collapse performance objective is not satisfied, a different structural upgrade strategy should be chosen. If the collapse performance objective is satisfied, expected losses should be estimated as described in the following step.

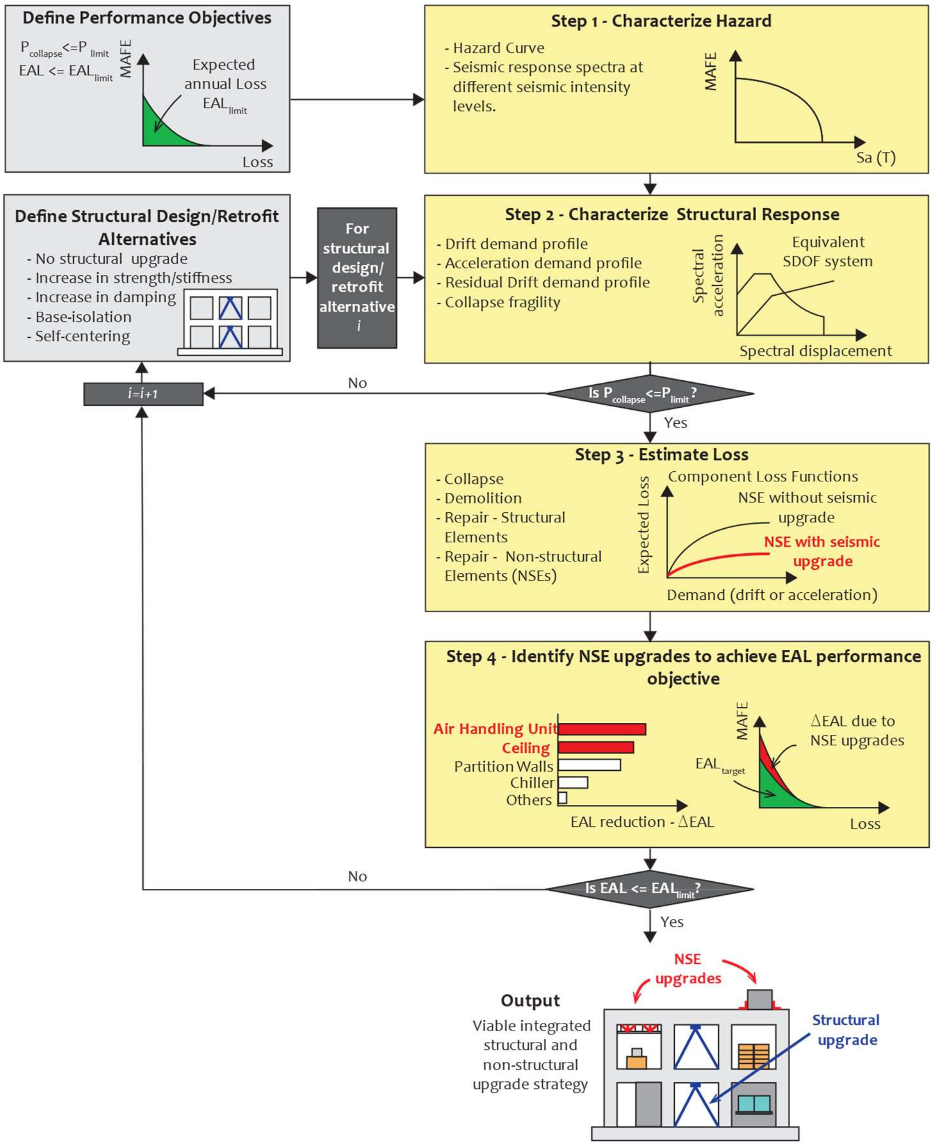

Figure 5.1 Overview of the proposed framework.

Step 3 - Estimate Loss. Component Loss-EDP functions are used to perform the loss estimation as described in Chapter 3. In this step, the expected annual loss of the building performance model with all non-structural elements not seismically upgraded is computed.

Step 4 - Identify NSE upgrades to achieve EAL performance objective. The expected annual loss reduction corresponding to the upgrade of each component is calculated using the component Loss-EDP function corresponding to its seismically-upgraded configuration. The non-structural element upgrades necessary to achieve the expected annual loss performance objective are identified in this step. If it is not possible to achieve the EAL performance objective even when all non-structural elements are assumed to be upgraded, an alternative structural upgrade strategy should be chosen, and the entire process repeated. If it is possible to achieve the EAL performance objective, the output of the framework is a viable combination of structural and non-structural upgrades to achieve both the collapse probability and the EAL performance objectives.

5.3 SEISMIC HAZARD AND STRUCTURAL RESPONSE ESTIMATION

The seismic demand at the site is characterized through the definition of the site hazard curve and uniform hazard response spectra corresponding to different seismic intensity levels. The seismic capacity of the building is represented by its pushover curve, which is described in terms of base shear and roof displacement. Using the Capacity-Spectrum approach [ATC, 1996], seismic demand and seismic capacity are combined to obtain the structural response at different seismic intensity levels, which is described in terms of the EDPs of Peak Interstorey Drift (PID), Peak Floor Acceleration (PFA), and Residual Drift (RD), as well as collapse fragility curve.

In order to estimate PID values corresponding to different intensity levels, structural capacity and seismic demand both need to be plotted in the spectral acceleration versus spectral displacement domain. The pushover curve needs to be converted into a capacity spectrum, which is defined in terms of spectral acceleration and spectral displacement of an equivalent single degree of freedom (SDOF) system. The conversion can be made using the following equations:

$$PF_1 = \left[\frac{\sum_{i=1}^{N}(w_i\varphi_{i1})/g}{\sum_{i=1}^{N}(w_i\varphi_{i1}^2)/g}\right] \tag{5. 1}$$

$$\alpha_1 = \frac{\left[\sum_{i=1}^{N}(w_i\varphi_{i1})/g\right]^2}{\left[\sum_{i=1}^{N}w_i/g\right]\left[\sum_{i=1}^{N}(w_i\varphi_{i1}^2)/g\right]} \tag{5. 2}$$

$$S_a = \frac{V/W}{\alpha_1} \tag{5. 3}$$

$$S_d = \frac{\Delta_{roof}}{PF_1 \varphi_{roof,1}} \qquad (5.\ 4)$$

where:

- PF_1 = modal participation factor for the first natural mode.
- α_1 = modal mass coefficient for the first natural mode.
- w_i/g = mass assigned to level i.
- ϕ_{i1} = amplitude of mode 1 at level i.
- N = level N, the level which is the uppermost in the main portion of the structure.
- V = base shear.
- W = bulding dead weight plus likely live loads.
- Δ_{roof} = roof displacement.
- S_a = spectral acceleration.
- S_d = spectral displacement.

The equivalent viscous damping at each intensity is calculated, and a reduced Acceleration-Displacement Response Spectrum (ADRS) is plotted. The performance point of the building at a given intensity is represented by the intersection between the reduced ADRS spectrum corresponding to that intensity and the capacity spectrum (Figure 5.2a). The relationship between the pushover curve and the capacity spectrum is used to calculate the roof displacements corresponding to each performance point. This requires users of the framework to assume a displaced shape of the structure based on the expected failure mechanism. Results from numerical pushover analysis, or analytical methods such as those suggested by Welch et al. [2014] and Del Vecchio et al. et al. [2020], or engineering judgement can be employed for this purpose. Once a displaced shape is assumed, interstorey drift values at each intensity level are calculated.

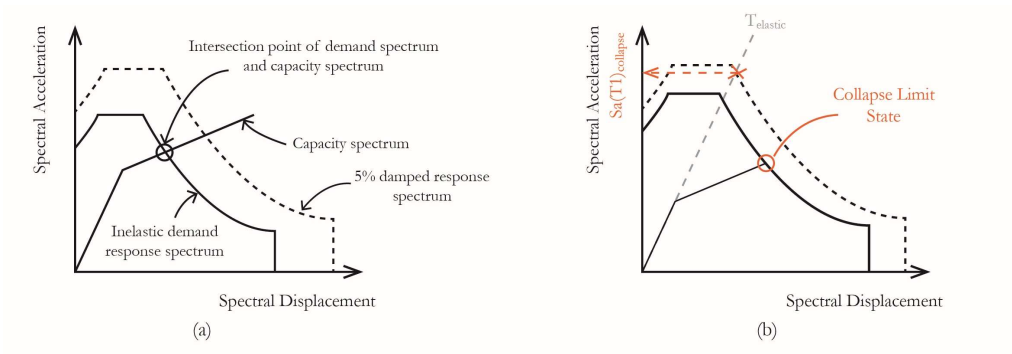

Figure 5.2 (a) Representation of building's capacity and demand in the spectral acceleration vs spectral displacement domain; (b) estimation of median spectral acceleration for collapse limit state.

The PFA values are estimated using empirical approximations provided in the FEMA P-58 documentation [FEMA P-58-1, 2018]. Specifically, the peak floor acceleration at the base of the building is equal to the peak ground acceleration *PGA*. At the other floor *i*, the median peak floor acceleration a_i^* is estimated using Equation 5.5.

$$a_i^* = H_{ai}(S,T,h_i,H) \times PGA \; for \; i = 2 \; to \; N+1 \tag{5. 5}$$

where *PGA* is the peak ground acceleration and $H_{ai}(S,T,h_i,H)$ is the acceleration corrector factor given by:

$$\ln(H_{ai}) = a_0 + a_1 T_1 + a_2 S + a_3 \frac{h_i}{H} + a_4 \left(\frac{h_i}{H}\right)^2 + a_5 \left(\frac{h_i}{H}\right)^3 \tag{5. 6}$$

$$for \; S \geq 1, i = 2 \; to \; N+1$$

The coefficients a_0 through a_5 are provided in the FEMA P-58 documentation and summarized in Table 5.1. *N* is the number of storeys. *S* is the strength ratio which can be calculated using Equation 5.7:

$$S = \frac{S_a(T_1)W}{V_{y1}} \tag{5. 7}$$

where $S_a(T_1)$ is the 5% damped spectral acceleration at the fundamental period of the building, V_{y1} is the estimated yield strength of the building in first mode response, and W is the total weight.

Table 5.1 Correction Factors for Floor Acceleration from the FEMA P-58-1 [2018]

Number of storeys	Frame Type	a0	a1	a2	a3	a4	a5
2-Story to 9-Story Buildings	Steel EBF[1]	0.66	-0.27	-0.089	0.075	0	0
	Steel SCBF[2]	1.15	-0.47	-0.039	-0.043	0.47	0
	Steel BRBF[3]	0.92	-0.3	-0.042	-0.25	0.43	0
	Moment Frame[4]	0.66	-0.25	-0.08	-0.039	0	0
	Wall	0.66	-0.15	-0.084	-0.26	0.57	0
10-Story to 15-Story Buildings	Steel EBF[1]	0.44	-0.27	-0.052	3.24	-9.71	6.83
	Steel SCBF[2]	0.63	-0.17	-0.046	3.52	-8.51	5.53
	Steel BRBF[3]	0.93	-0.191	-0.057	1.67	-4.6	3.06
	Moment Frame[4]	0.34	-0.25	-0.062	2.86	-7.43	5.1
	Wall	-0.13	-0.15	-0.1	7.79	-17.52	11.04

[1] Steel EBF = Steel eccentrically braced frames

[2] Steel SCBF = Steel special concentrically braced frames

[3] Steel BRBF = Steel buckling-restrained braced frames

[4] Moment Frame = Steel and reinforced concrete special moment-resisting frames

The equations provided in the FEMAP-58-1 [2018] are also used to estimate the residual drift at different intensity levels (Equations 5.8, 5.9 and 5.10).

$$\Delta_r = 0 \;\; for\; \Delta \leq \Delta_y \tag{5. 8}$$

$$\Delta_r = \mathbf{0.3}(\Delta - \Delta_y) \;\; \boldsymbol{for}\; \Delta_y < \Delta < \mathbf{4}\Delta_y \tag{5. 9}$$

$$\Delta_r = (\Delta - \mathbf{3}\Delta_y) \;\; \boldsymbol{for}\; \Delta \geq \mathbf{4}\Delta_y \tag{5. 10}$$

where Δ_r is the median residual drift ratio expressed as a function of the median story drift ratio Δ and median story drift ratio calculated at yield Δ_y.

In addition to EDP values, the collapse fragility curve of the building needs to be calculated. Several approaches have been proposed in the literature for analytical vulnerability assessment [D'Ayala et al., 2015]. A simplified approach is used in this framework to estimate the median and dispersion of the collapse fragility curve. The proposed approach requires that the collapse limit state is identified on the pushover curve. Results from numerical pushover analysis or analytical considerations based on a P-delta stability coefficient and deformation/drifts limits provided for the collapse prevention limit state [Priestley et al., 2007] can be used to indentify the collapse limit state. The intensity that will cause this collapse limit state to be exceeded can be identified in the spectral displacement-spectral acceleration domain by estimating the ADRS spectrum that intersects the collapse limit state point on the bilinear pushover curve. This ADRS spectrum represents the inelastic demand on the building, and it must be scaled to obtain the 5% elastic response spectrum (Figure 5.2b). The elastic spectral acceleration at the first period of the structure estimated from the elastic ADRS spectrum is assumed as the 50th percentile value of the collapse fragility curve. Following the FEMA P-58-1 [2018] guidelines, a log-normal distribution is assumed for collapse and a large dispersion equal to 0.6 is assumed as suggested for regular structures when a judgement-based collapse fragility is used.

5.4 LOSS ESTIMATION AND IDENTIFICATION OF NON-STRUCTURAL UPGRADES TO ACHIEVE EAL PERFORMANCE OBJECTIVE

Once the structural response is characterized, the loss estimation is performed using the approach proposed in Chapter 3. Structural and non-structural elements in the performance model are defined by Loss-EDP functions and, for each element, expected losses corresponding to each seismic intensity are calculated using Equation 3.7. Non-structural elements are characterized by two Loss-EDP functions corresponding to their seismically and not-seismically upgraded configurations. First, the expected annual loss of the building considering all non-structural elements not-seismically upgraded is calculated by integrating losses over all considered seismic intensities. Then, the expected annual loss reduction ΔEAL_j that could be obtained by upgrading each non-structural element in the building performance model is calculated using Equation 5.11:

$$\Delta EAL_j = EAL_{j,NSU} - EAL_{j,SU} \tag{5. 11}$$

$$\Delta EAL_j = \int E\left[L_{j,NSU}|IM\right]\left|\frac{d\lambda}{dIM}\right| dIM - \int E\left[L_{j,SU}|IM\right]\left|\frac{d\lambda}{dIM}\right| dIM \tag{5. 12}$$

where $EAL_{j,SU}$ and $EAL_{j,NSU}$ are the expected annual losses due to damage to component j in its seismically upgraded and not-seimcally upgraded configuration; $E[L_{j,NSU}|IM]$ and $E[L_{j,SU}|IM]$ are the expected losses due to the component j in its not-sesimically upgraded and seismically upgraded configuration for a seismic intensity IM; and λ is frequency of exceedance of a ground motion intensity IM.

Non-structural element upgrades are prioritized according to their expected annual loss reduction ΔEAL_j, and non-structural upgrades necessary to achieve the EAL performance objective are identified.

5.5 REVIEW AND DISCUSSION

A framework for identifying viable combinations of structural and non-structural upgrade strategies has been presented in this chapter. A viable upgrade strategy is defined based on the collapse probability and expected annual loss performance objectives that are set at the beginning of the framework. For each considered structural retrofit/design strategy, an equivalent Single Degree of Freedom (SDOF) approximation is used to perform the structural analysis and component Loss-EDP functions tailored to the seismic design rating of each component are used to perform the loss analysis. After identifying the structural upgrade strategies that satisfy the collapse performance objective, the framework enables to identify non-structural upgrades to achieve the expected annual loss performance objective. The framework's output is a viable building upgrade configuration which is characterized by a probability of collapse lower than the probability of collapse performance objective and an EAL lower than the EAL performance objective.

The next chapter will be dedicated to the application of the proposed framework to a six-storey moment resisting frame. The validation of the framework will be performed using time history analyses to estimate the structural response and the FEMA P-58 methodology to perform the loss estimation.

6.PUSHOVER-BASED FRAMEWORK APPLICATION TO A SIX-STOREY STEEL MOMENT RESISTING FRAME

6.1 CHAPTER OVERVIEW

In this chapter, the pushover-based framework to assess integrated structural and non-structural upgrade strategies presented in Chapter 5 is applied to a six-storey moment resisting frame. The results of the framework are validated using time history analyses and the FEMA P-58 methodology [FEMA P-58-1, 2018].

6.2 PUSHOVER-BASED FRAMEWORK APPLICATION

The proposed pushover-based framework is applied to the six-storey steel moment resisting frame presented in Chapter 4. As discussed in Chapter 4, the six-storey archetype building was adapted from the FEMA 440 document [FEMA, 2005]. It has a total floor area of 2007 m² and the seismic force-resisting system is composed of moment-resisting frames along the building's perimeter, while interior frames are designed to carry only gravity loads. It comprises three bays in the North-South direction and six bays in the East-West direction. For this study, only the North-South direction was considered (Figure 4.1). The modeling of the frame was implemented in the OpenSees software [McKenna *et al.*, 2010] using *BeamWithHinges* elements and the *Steel02* material to model beams and columns of each moment-resisting frame. The interior gravity frames were modeled using a leaning gravity column to account for P-Delta effects. The computed fundamental period of the archetype building is 1.3 s. This period is slightly different compared to the one computed in Chapter 4 and listed in Table 4.1. This is because a different assumption was made for the modeling of the panel zones of the beam-column connections of the archetype building. In Chapter 4, the panel zones were modeled using the procedure proposed by Gupta and Krawinkler [1999], in which the panel zone is composed of a series of rigid pin-connected elements and all the shear deformations are concentrated in a plastic rotational spring in one of the corners. In this chapter, to speed up the analyses without losing too much accuracy, a simplified approach is used and the panel zones of the beam-column connections are assumed to be stiff and strong enough to avoid any panel shear deformation and yielding under strong earthquakes. Rigid elements at the end of beams and columns are used to model the panel zones in OpenSees. Using this second assumption, all hysteretic energy is dissipated through plastic hinging in the beams and the columns, which is the most critical condition for the inelastic curvature demand on the welded beam-to-column joints.

6.2.1 Performance objectives and structural retrofit alternatives

A probability of collapse and an Expected Annual Loss (EAL) performance objective must be set at the beginning of the framework. For this study, a probability of collapse at MCE (Maximum Credible Earthquake) of 20% and an EAL of 0.2% of the building cost were assumed for illustration as performance objectives. In addition to the building in its original (as-built) configuration, two retrofit alternatives are assessed. The first retrofit strategy consists of introducing a chevron-braced frame in the central bay of each moment-resisting frame with hysteretic energy-dissipating devices at one end of the bracing members. In the second investigated retrofit strategy, linear viscous-type energy-dissipating devices at one end of the bracing members are used instead of hysteretic energy-dissipating devices.

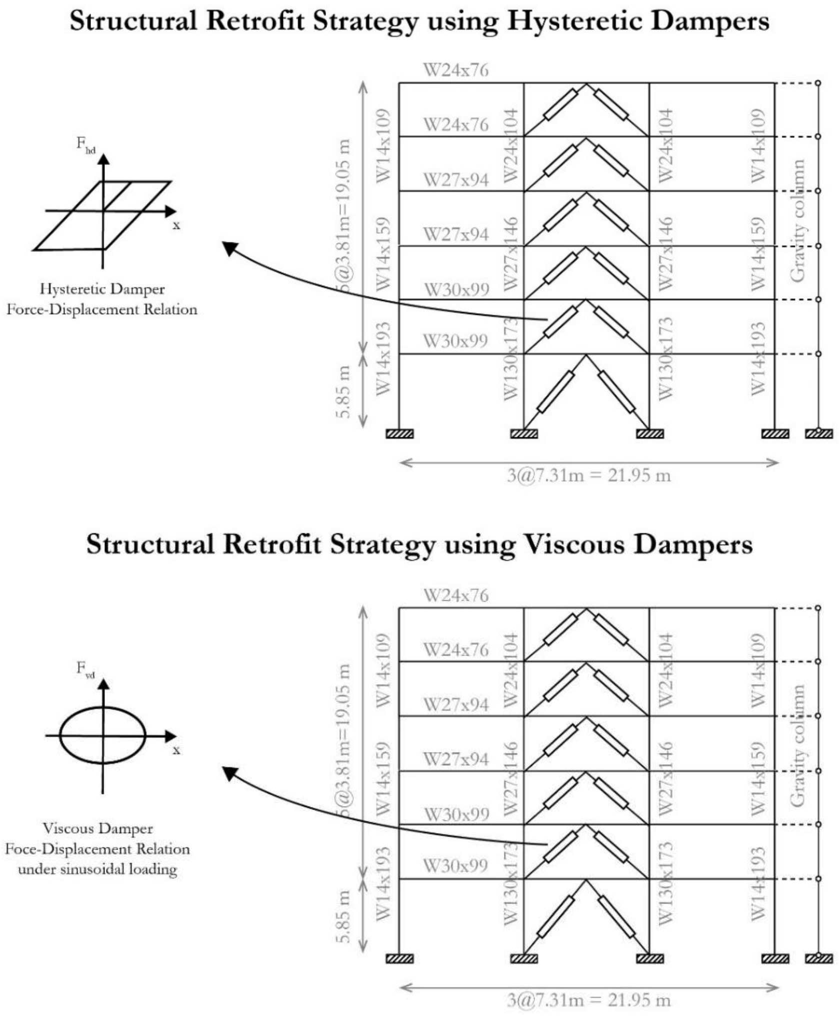

Figure 6.1 Location of added braces and energy-dissipating devices for the two investigated structural retrofit strategies.

The use of hysteretic dampers is expected to provide an increase in the building's strength and stiffness, which is represented in the proposed framework by a change in the pushover curve of the building. The change in the pushover curve was estimated via numerical pushover analysis by modelling the hysteretic energy dissipating devices in Opensees. Two parameters must be defined for modelling the hysteretic dampers: the section of the braces and the activation load of the hysteretic dampers, which are assumed to have a rigid-plastic hysteretic behaviour. The preliminary design of these parameters was performed using the methodology proposed by Filiatrault and Cherry [1988, 1990]. The initial configuration of the braces and activation loads of the hysteretic dampers obtained using this preliminary design methodology are listed in Table 6.1. An initial configuration of the braces was assumed using braces with square hollow structural sections with the largest cross-section that can be accommodated within the building dimensions (i.e. cross-section dimensions not larger than the column flanges width). The modal analysis of the braced frame was conducted, and a braced period of 0.66 s was obtained. The ratio between the fundamental period of the braced frame T_b and the fundamental period of the unbraced frame T_u is equal to 0.51, which was considered close enough to the recommendation of $Tb/Tu<0.4$ by Filiatrault and Cherry [1988, 1990].

Table 6.1. Preliminary design of the structural retrofit strategy using hysteretic dampers

Level	Brace sections	Activation Load [KN]	Tb [s]	Tb/Tu
1	HSS 14x14x5/8"	1350	0.66	0.51
2		1100		
3	HSS 12x12x5/8"	1100		
4				
5	HSS 10x10x5/8"	1100		
6				

The use of viscous dampers, on the other hand, does not produce any change in the building pushover curve but provides a reduction in the seismic demand of the building due to the additional source of viscous damping. As the pushover curve of the building remains the same, for this retrofit strategy it is not necessary to implement the numerical modeling of the dampers. The only information required to apply the proposed framework is the value of the viscous damping ratio in the first mode of vibration, which is expected to be provided by the viscous energy-dissipating devices. For this preliminary design phase, a viscous damping ratio of 20% was assumed for illustration.

6.2.2 Steps 1 and 2: seismic hazard and structural response estimation

The hazard curve at the site and the site uniform hazard response spectrum at different seismic intensity levels were obtained using the USGS (United States Geological Survey)

Uniform Hazard Tool [USGS, 2014]. For all considered structural upgrade strategies, the structural response was estimated using the capacity spectrum-based structural response estimation procedure described in Chapter 5. The pushover curve of the building was obtained through numerical analysis (Figure 6.2). To capture the brittle failure of the welded beam-to-column connections, a flexural strength degradation model was introduced at the ends of the beam and column elements. A plastic rotation limit of 0.03 radians was assumed as the failure criterion for all steel beam and column elements. When the plastic rotation limit is exceeded, the strength of the elements reduces to 1% of the yield moment, which is close enough to zero for engineering purposes. This assumption is consistent with the brittle failure experienced by the beam-column connections of moment-resisting frames designed before the 1994 Northridge earthquake.

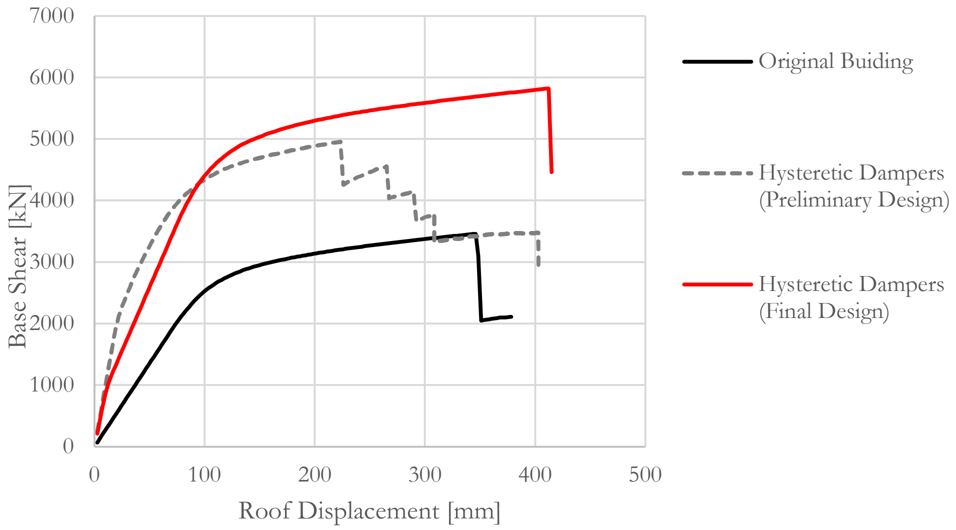

Figure 6.2 Pushover curve of the original building and of the building retrofitted using hysteretic dampers with the preliminary and final design properties.

As illustrated in Figure 5.1, after Step 2, the probability of collapse at MCE is compared with the collapse performance objective. If the collapse performance objective is not satisfied, a different structural upgrade strategy should be chosen. The probability of collapse at MCE obtained for the investigated structural configurations is listed in Table 6.2. For the original building, a probability of collapse at MCE of 62% was obtained. This high probability of collapse could be expected due to the poor performance of the brittle welded beam-to-column connections and the soft storey at the first floor. For the building retrofitted with hysteretic damping devices, a probability of collapse at MCE of 44% was obtained. The probability of collapse of the retrofitted structure is reduced compared to the original building. However, as the probability of collapse is still above the probability of collapse performance objective of 20%, the design properties of the hysteretic dampers were modified to improve the structural performance of the building. After some iterations, the hysteretic properties in Table 6.3 were selected: hysteretic dampers on the top two floors were removed; braces and hysteretic dampers were installed also in the lateral bays of the first floor; the activation loads of the hysteretic dampers on the first floor were increased and activation loads on the other floors were reduced. This hysteretic damper configuration achieves a probability of collapse equal to 22.4%. This probability of collapse is still slightly above the probability of collapse performance objective but is considered acceptable for this illustrative example. More iterations on the hysteretic damper properties could be conducted to obtain a probability of collapse below the target performance objective. However, this is outside of the scope of this study, whose main intent is to illustrate the framework application and validate its results. For illustration purposes, the hysteretic configuration listed in Table 6.3 was judged to be close enough to the target performance objective and the loss estimation step described in the next paragraph was applied.

For the building retrofitted with viscous dampers, using the procedure described in Chapter 5 and accounting for the additional source of viscous damping, a probability of collapse at MCE of 40% was found. To further reduce this probability of collapse, a larger value of viscous damping ratio equal to 35% was assumed, which was considered as the maximum viscous damping ratio that can be achieved economically with currently available viscous dissipation devices. The new assumed viscous damping ratio reduced the probability of collapse at MCE to 30%. This value is still above the collapse probability performance objective and a different structural upgrade strategy should be defined to obtain a viable combination of structural and non-structural upgrades. However, for this study, although this structural upgrade strategy was not able to achieve the collapse probability performance objective, the loss estimation step was performed to assess the effect of using viscous dampers on building seismic losses.

Table 6.2. Probability of collapse at MCE of the investigated structural configurations

	Probability of collapse at MCE
Original building	62.0%
Building with hysteretic dampers preliminary design	44.0%
Building with hysteretic dampers final design	22.4%
Building with viscous dampers preliminary design	40.0%
Building with viscous dampers final design	30.0%

Table 6.3. Final design of the structural retrofit strategy using hysteretic dampers

Level	Brace sections central bay	Brace sections lateral bays	Activation Load [KN]	Tb [s]	Tb/Tu
1	HSS 10x10x5/8"	HSS 10x10x5/8"	1500	0.71	0.55
2		-	500		
3	HSS 10x10x5/8"	-	300		
4					
5	-	-	-		
6					

6.2.3 Steps 3 and 4: Loss estimation and identification of non-structural upgrades to achieve EAL performance objective

Using Loss-EDP component functions, as described in Chapter 5, the expected annual loss of the building retrofitted with hysteretic and viscous dampers was computed. The non-structural elements in Table 4.3 of Chapter 4 were included in the building performance model. For the Loss-EDP functions of the structural elements, fragility and consequence functions of the FEMA P-58 database corresponding to Pre-Northridge connections were used (components B1035.042 and B1035.052 in the FEMA P-58 database).

The EAL values obtained for the two investigated retrofit strategies are listed in Table 6.4. When all non-structural elements in the building are assumed to be not seismically designed, the EAL of the building retrofitted with hysteretic dampers was found to be equal to 0.28% of the building cost and the EAL of the building retrofitted with viscous dampers was found to be equal to 0.14% of the building cost. The installation of hysteretic dampers produces an increase in the stiffness of the building and thus, the drift demand on non-structural elements reduces but the acceleration demand increases. Therefore, an increase in damage due to acceleration-sensitive non-structural elements can be expected when hysteretic dampers are used. On the other hand, for the building retrofitted with viscous dampers, both losses due to drift-sensitive and acceleration-sensitive non-structural elements are expected to be reduced, which is consistent with the lower value of EAL obtained for the building retrofitted with viscous dampers compared to the building retrofitted with hysteretic dampers. Following the framework steps in Figure 5.1, a check is performed after Step 4 to establish if the EAL performance objective is achieved. If it is not possible to achieve the EAL performance objective even when all non-structural elements are assumed to be upgraded, an alternative structural upgrade strategy should be chosen, and the entire process repeated. If it is possible to achieve the EAL performance objective, the output of the framework is a viable combination of structural and non-structural upgrades to achieve both the collapse probability and the EAL performance objectives. The EAL of the building retrofitted with viscous dampers is below the EAL performance objective, thus non-structural upgrades are not required to achieve the target performance.

Table 6.4. EAL of the investigated structural upgrade strategies

	EAL [% BC]
Assumed Target	0.20%
Building with hysteretic dampers	0.28%
Building with hysteretic dampers and critical NSE upgraded	0.18%
Building with viscous dampers	0.14%

On the other hand, for the building retrofitted with hysteretic dampers, the Loss-EDP functions of the seismically designed non-structural elements are used to calculate the EAL reduction provided by each non-structural upgrade. The EAL reduction is calculated using the procedure described in Chapter 3 to perform Step 4 of non-structural upgrade assessment framework. Non-structural upgrades are then prioritized according to the EAL reduction that they could provide (Figure 6.3). For the building retrofitted with hysteretic dampers, the upgrades of the chiller and air handling units were found to reduce the EAL to 0.18% of the building cost, which is below the EAL performance objective. Therefore, the chiller and air handling unit are identified as the critical non-structural upgrades to be implemented to achieve the EAL performance objective.

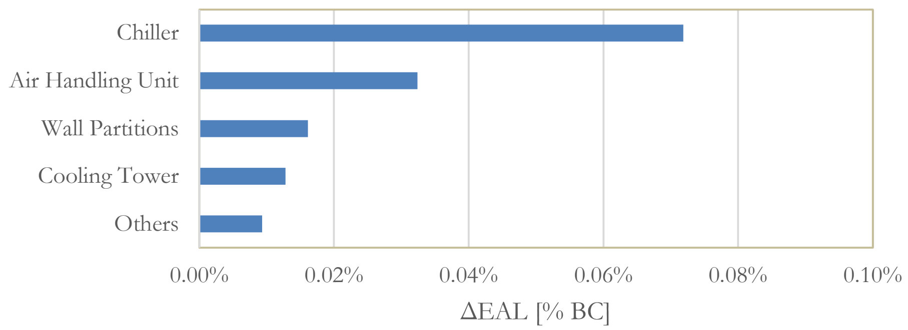

Figure 6.3 EAL reduction due to non-structural element upgrades in the building retrofitted with hysteretic dampers.

6.3 FRAMEWORK VALIDATION

The results of the framework were validated using time history analyses and the full P-58 methodology (using the PACTtool). The numerical modeling of the building retrofitted with hysteretic dampers was performed using the hysteretic damper properties in Table 6.3. For the building retrofitted with viscous dampers, the properties of the viscous dampers to achieve the assumed 35% value of viscous damping ratio were defined as described below.

For this study, linear viscous dampers were used and the linear damping constant C_L was defined for each damper along the building height. As the introduction of linear viscous damping in typical structures yields a stiffness proportional damping matrix, a practical approach can be followed for the preliminary design of C_L [Christopoulos and Filiatrault, 2022]. First, a fictitious undamped brace structure was defined. Fictitious springs $\hat{k}_0^n$ were added at the proposed locations of the viscous dampers, distributed according to the interstorey lateral stiffness to ensure that the same fundamental mode shape of the original building was obtained for the fictitiously braced structure. Given the fundamental period of the original building T_1, the required fundamental period of the fictitiously braced frame $\hat{T}_1$ corresponding to a desired first mode viscous damping ratio ξ_1 can be computed using Equation 6.1:

$$\hat{T}_1 = \frac{T_1}{\sqrt{2\xi_1 + 1}} \tag{6. 1}$$

The stiffness of the fictitious springs $\hat{k}_0^n$ that yields to a fundamental period of the fictitiously braced structure equal to $\hat{T}_1$ can be obtained using Equation 6.2:

$$\hat{k}_0^n = \frac{\hat{k}_{0tr}^n}{\left(\frac{T_1^2 - \hat{T}_{1tr}^2}{T_1^2 - \hat{T}_1^2}\right)\left(\frac{\hat{T}_1^2}{\hat{T}_{1tr}^2}\right)} \tag{6. 2}$$

where $\hat{k}_{0tr}^n$ is an initial trial value of the stiffness coefficient of the fictitious spring and $\hat{T}_{1tr}$ is the corresponding trial value of the fundamental period of the fictitious braced frame with these trial fictitious spring constants. After the fictitious spring constants are computed, the linear damping constant C_L can be obtained:

$$C_L = \frac{T_1}{2\pi}\hat{k}_0 \tag{6. 3}$$

The preliminary design results of the linear damping constants are summarized in Table 6.5. The preliminary design values of C_L were used to model the viscous energy-dissipating devices in the central bay of the moment-resisting frame. In order to verify that the assumed damping ratio of 35% was achieved, the free vibration of the structure was simulated. The logarithmic decrement was used to calculate the damping ratio value in the numerical model and the design of the linear damping constants was slightly modified to obtain the assumed damping ratio value of 35%. The final linear damping constants C_L used in the numerical model and corresponding to a viscous damping ratio of 35% are listed in the last column of Table 6.5.

Table 6.5. Design of the structural retrofit strategy using viscous dampers

Storey	$\hat{k}_{0tr}$ [kN/mm]	$\hat{T}_{1tr}$ [sec]	$\hat{k}_{0\ \text{hoizontal}}$ [kN/mm]	$\hat{k}_{0\ \text{sping direction}}$ [kN/mm]	$C_{L\ \text{preliminary design}}$ [kN.s/mm]	$C_{L\ \text{final design}}$ [kN.s/mm]
1	16.2	1.2	58.6	95.2	19.7	20.0
2	22.2		80.5	83.9	17.4	18.0
3	19.8		71.8	74.8	15.5	18.0
4	17.9		64.7	67.5	14.0	18.0
5	13.5		48.7	50.8	10.5	16.0
6	10.4		37.7	39.3	8.1	16.0

The building retrofitted with hysteretic dampers was modelled in Opensees using *truss* elements and *steel01* material to simulate the hysteretic behaviour of the dampers. For the building retrofitted with viscous dampers, *twoNodeLink* elements were used along with the *ViscousDamper* material to simulate the behaviour of the linear viscous dampers.

Time history analyses were used to obtain the structural response input data required to apply the FEMA P-58 methodology. The same procedure described in Chapter 4 was used to scale the 44 records of the FEMA P-695 far-field set at the different intensity levels illustrated in Figure 4.7. The probability of collapse at MCE and the EAL obtained using the FEMA P-58 methodology are listed in Table 6.6 and compared with the results of the proposed framework. The EAL comparison is illustrated in Figure 6.4.

Table 6.6. Comparison between the FEMA P-58 methodology and the proposed framework

	Proposed Framework		FEMA P-58 (PACT)	
	Probability of collapse at MCE	EAL [%BC]	Probability of collapse at MCE	EAL [%BC]
Original Building	62%	0.46%	53%	0.39%
Building with hysteretic dampers	22%	0.28%	28%	0.29%

Building with hysteretic dampers and critical NSE upgraded	22%	0.18%	28%	0.19%
Building with viscous dampers	30%	0.14%	29%	0.17%

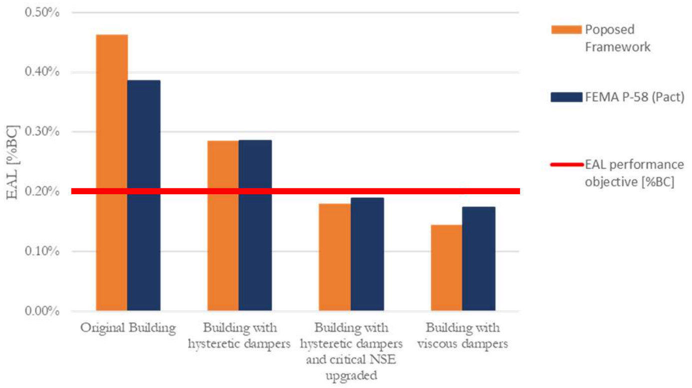

Figure 6.4 EAL comparison between the FEMA P-58 methodology and the proposed framework.

The largest difference between the collapse probability estimated using time history analyses and the proposed framework was obtained for the original building. Using the proposed framework, a probability of collapse at MCE equal to 62% was obtained, while the use of time history analyses led to a collapse probability at MCE of 53%. The probability of collapse at MCE was overestimated by the proposed framework for the original building, while it was underestimated for the building retrofitted with hysteretic energy-dissipation devices. A similar probability of collapse was obtained for the building retrofitted with viscous dampers using the two approaches. Considering the very simplified approach that is used to estimate the collapse fragility curve in the proposed framework, such discrepancies could be expected. However, the fact that the probability of collapse was underestimated for some structural configurations and overestimated for others is not ideal as the main purpose of the framework application is to compare different structural upgrade configurations. Moreover, as the framework is intended as a preliminary design tool, an overestimation of the collapse probability would have been preferable to an underestimation, which was obtained for the building retrofitted with hysteretic dampers. As future research development, other methodologies for simplified collapse fragility curve estimation proposed in the literature might be investigated or new ones developed to improve this aspect of the proposed framework.

In terms of EAL comparison, similar results were obtained for all the investigated structural configurations. The proposed framework was able to capture which combinations of structural and non-structural upgrades were able to achieve an EAL below the EAL performance objective and which ones were not. Furthermore, the framework was able to identify critical non-structural upgrades to improve the building's performance and achieve the EAL performance objective. Note that these results were obtained with the proposed framework with only a small fraction of the effort required to implement the full FEMA P-58 Methdology.

6.4 REVIEW AND DISCUSSION

In this chapter, the proposed pushover-based framework to assess structural and non-structural upgrade strategies was applied to a six-storey moment resisting frame. A probability of collapse at MCE of 20% and an EAL of 0.2% of the building cost were assumed as performance objectives and two structural upgrade configurations using hysteretic and viscous dampers were assessed. It was found that both structural upgrade strategies were unable to achieve the probability of collapse performance objective.The EAL of the building retrofitted with viscous dampers was found to be below the EAL performance objective without the need for non-structural element upgades; the EAL of the building retrofitted with hysteretic dampers, on the other hand, was found to be above the EAL performance objective and the chiller and air handling unit were identified as critical non-structural upgrades to achieve the target EAL performance.

The results of the framework were validated using time history analyses and the FEMA P-58 methodology (using the PACT tool). The largest difference in the probability of collapse at MCE was obtained for the original building, for which a probability of collapse at MCE of 62% was obtained using the proposed framework and a probability of collapse of 53% was obtained using time history analysis. For the building retrofitted with hysteretic dampers, the probability of collapse was underestimated using the proposed framework, while similar results were obtained for the building retrofitted with viscous dampers. Similar values of EAL were obtained using the proposed framework and the FEMA P-58 methodology and the framework was able to identify the critical non-structural elements to upgrade to achieve the EAL performance objective for the building retrofitted with hysteretic dampers.

The results of this study show the potential of the framework as a simplified tool that can help engineers to conduct a preliminary assessment and reduce the number of potential integrated structural and non-structural upgrade strategies to investigate using a more rigorous assessment methodology. The reduced computational effort required to estimate the structural response and perform the loss estimation makes the proposed framework very efficient in a preliminary design phase to assess multiple upgrade strategies when limited resources are available. The use of component Loss-EDP was found to be particularly effective for quantifying the seismic risk associated with NSEs and communicating the benefit of upgrading them to stakeholders at an early stage of the design process. Although the framework's application to a case study building yielded promising results, the proposed framework still needs to be more comprehensively validated by investigating more archetype buildings and structural upgrade strategies.

7.CONCLUSIONS

7.1 GENERAL SUMMARY

Recent seismic events have demonstrated the importance of harmonizing the seismic performance of structural and non-structural elements. Even if a good structural performance is achieved during an earthquake, damage to non-structural elements can pose a risk to life-safety and produce large monetary losses and loss of building functionality. When designing upgrade strategies to improve the overall performance of a building, it is important to consider the relationship between structural and non-structural performance. The seismic response of a structure represents the seismic demand on its non-structural elements. Therefore, the benefit of a structural upgrade may be reduced if the change in the structural response produces an increased seismic demand on the non-structural elements. On the other hand, the performance of non-structural elements and the benefit of non-structural upgrades become less relevant if a poor structural performance is achieved. An important step forward towards the harmonization of structural and non-structural performance was the development of the performance-based earthquake engineering (PBEE) framework by the Pacific Earthquake Engineering Research Center (PEER) and implemented in the FEMA P-58 methodology. Using the FEMA P-58 methodology, the effect of structural and non-structural upgrades on the building's performance can be assessed. However, when multiple combinations of structural and non-structural upgrades are considered, the use of the FEMA P-58 methodology to identify optimal seismic upgrade combinations becomes impractical as a trial-and-error approach should be used, which requires an extensive computational effort. Although many advancements have been made in the earthquake engineering field for the harmonization of structural and non-structural performance, there remains a need for simplified procedures to assess the impact of structural and non-structural upgrade combinations at a preliminary design phase, when limited resources are available. The objective of this study was to develop frameworks that can help harmonize the seismic performance of structural and non-structural elements and enhance the transparency of the process for identifying viable combinations of structural and non-structural upgrades. The frameworks proposed in this thesis are intended to help designers in the preliminary phase of the decision-making process to identify the key drivers that affect non-structural upgrade impact on seismic loss reduction, prioritize non-structural element upgrades and identify viable combinations of structural and non-structural upgrade strategies.

After an Introduction presented in Chapter 1, Chapter 2 presented a review of different methodologies that have been developed for the seismic risk assessment of buildings and provided a discussion on the role of non-structural elements. Component-based and storey-based loss estimation approaches were discussed and recent optimization frameworks to identify optimal combinations of structural and non-structural upgrades were presented. The concept of storey DV-EDP functions, which directly relate economic losses (Decision Variable, DV) to structural response parameters (Engineering Demand Parameter, EDP) was introduced in Chapter 2. In Chapter 3, a framework that applies the concept of DV-EDP functions at a component level was proposed for the preliminary assessment of non-structural upgrades. The proposed framework named “non-structural upgrade assessment framework” is composed of four sequential steps characterized by different levels of sophistication of the input data. The first three steps are used to reduce the number of potential non-structural upgrades by removing upgrades that are not likely to produce a significant improvement to building performance. The fourth step is used to prioritize non-structural upgrades that remain after Step 3, using as the benefit-cost ratio as a metric. An overview of the Non-Structural upgrade assessment Excel Tool to implement the framework was also provided in Chapter 3. In Chapter 4, the non-structural upgrade assessment framework was applied to three steel moment-resisting frames with three, six and nine storeys. The results of the proposed framework were validated using the FEMA P-58 methodology. Chapter 5 presented a Pushover-based framework to assess integrated structural and non-structural upgrade combinations. Target performance objectives are defined in the framework in terms of probability of collapse and expected annual loss. An equivalent Single Degree of Freedom (SDOF) approximation is used to perform the structural analysis and component DV-EDP functions tailored to the seismic design rating of each component are used to perform the loss analysis. The framework application to a six-storey moment resisting frame was discussed in Chapter 6. Two structural upgrade strategies were investigated, which are 1) the use of hysteretic dampers, and 2) the use of linear viscous dampers. The results of the framework were validated using time history analyses and the FEMA P-58 methodology.

7.2 MAIN FINDINGS OF THE DISSERTATION

The most significantfindings of this thesis are summarized in this section.

7.2.1 Chapters 3 and 4: Non-structural upgrade assessment framework development and application

- The use of four steps which require different levels of amount and sophistication of the input data was found to be very practical and efficient to obtain relevant information on non-structural element upgrades without necessarily investing a large number of resources. In order to apply the FEMA P-58 methodology, all information on the building performance model and structural response are required. On the other hand, the first steps of the proposed framework can be applied without the need to implement the model of the building and run structural analyses. For the archetype buildings presented in Chapter 4, for example, upgrades to components such as pipings and HVAC ducts were found to not contribute much to seismic loss reduction from the first step of the framework, when only general information on the building such as number of floors, floor area and occupancy were available.

- The proposed framework can be easily implemented in an Excel spreadsheet (provided in Appendix C) and the computational effort required to apply it was found to be much lower than the one required to apply the FEMA P-58 methodology to assess non-structural upgrade impact.
- The use of component Loss-EDP functions, which directly relate expected loss with EDP values, was found to improve the transparency of the non-structural upgrade assessment process as it helped understand how much the impact of an upgrade was sensitive to change in the structural response. For instance, in Chapter 4, the Loss-EDP functions of the sprinkler piping, the chiller and the suspending ceilings in the three-storey building were examined and it was possible to determine that the upgrade of sprinkler piping would not have made a significant contribution to seismic loss reduction in the EDP range of interest; the upgrade of chiller could have made a significant contribution for PFA values lower than 0.5 g; and the upgrade of suspending ceilings would have provide its maximum contribution to seismic loss reduction for values of PFA above 2 g. This is an important advantage of the framework compared to the FEMA P-58 methodology because, using the FEMA P-58 methodology, it is not straightforward to understand why an upgrade is more important than another and how much the non-structural upgrade impact might be sensitive to change in the structural response.
- The framwork validation conducted using the FEMA P-58 methodology showed that the framework was able to identify the most critical NSE upgrades to maximize the benefit-cost ratio of the investment.
- From the non-structural upgrade assessment framework application to three archetype buildings, the upgrade of the chiller was found to have the highest benefit-cost ratio for all the archetype buildings, followed by the cooling tower and the low voltage switchgear. For the six-storey and nine-storey building, only the upgrade of the chiller was found to have a benefit-cost ratio larger than 1, which indicates a viable upgrade (the loss reduction produced by the upgrade exceeds the upgrade implementation cost). For the three-storey building, all non-structural upgrades were found to have a benefit-cost ratio lower than 1.

7.2.2 Chapters 5 and 6: Pushover-based framework to assess integrated structural and non-structural upgrade strategies development and application

- A substantial reduction of the computational time required to identify viable combinations of structural and non-structural upgrade strategies was found when applying the proposed framework compared to the full FEMA P-58 Methodology.

- The results of the framework were validated using time history analyses and the FEMA P-58 methodology. Similar values of EAL were obtained using the proposed pushover-based framework and the FEMA P-58 methodology. For the building retrofitted with hysteretic dampers, the framework was able to identify the critical non-structural element upgrades to achieve the EAL performance objective. For the original building, the proposed framework overestimated the probability of collapse. A probability of collapse at MCE of 62% was obtained using the proposed framework and a probability of collapse of 53% was obtained using time history analyses. For the building retrofitted with hysteretic dampers, the probability of collapse was underestimated using the proposed framework. The collapse probability was equal to 22% using the proposed framework and 28% using time history analyses. Similar results were obtained for the building retrofitted with viscous dampers, for which a probability of collapse at MCE equal to 30% and 29% were found using the proposed framework and time history analyses, respectively.
- The proposed pushover-based framework was applied to a six-storey moment resisting frame, assuming a probability of collapse at MCE of 20% and an EAL of 0.2% of the building cost as target performance objectives. Two structural strategies using hysteretic and viscous dampers were assessed. A probability of collapse at MCE equal to 22.4% was obtained for the building retrofitted with hysteretic dampers and a probability of collapse at MCE equal to 30% was obtained for the building retrofitted with viscous dampers. Therefore, both retrofit strategies were not able to achieve the target collapse probability performance objective. The EAL of the building retrofitted with viscous dampers was found to be equal to 0.14% of the building cost, which is below the EAL performance objective. Therefore, non-structural element upgades were not necessary to achieve the EAL performance objective. On the other hand, the EAL of the building retrofitted with hysteretic dampers, was found to be equal to 0.28% of the building cost, which is above the EAL performance objective. The upgrades of the chiller and the air handling unit were identified as the critical non-structural upgrades to achieve the target EAL performance.

7.3 INNOVATIVE ASPECTS

The innovative aspects of this thesis are listed below:

- Development of a framework to assess non-structural element upgrades which comprises sequential steps corresponding to different amounts and sophistication of required input data.
- Development of non-structural element Loss-EDP functions tailored to the element seismic design rating.

- Development of an Excel tool to implement the non-structural upgrade assessment framework.
- Development of a pushover-based framework for the preliminary assessment of different combinations of structural and non-structural upgrade strategies.

7.4 FUTURE WORK

This section discusses some aspects of the two proposed frameworks that require further investigation, as well as some potential use of the framework for extending its scope.

- In the case study application of the pushover-based framework, the probability of collapse was underestimated for some structural configurations and overestimated for others. This is not ideal because the main purpose of the framework is to compare different structural upgrade configurations. Moreover, as the framework is intended to be used in a preliminary design phase, an overestimation of the collapse probability would have been preferable to an underestimation, which was obtained for the building retrofitted with hysteretic dampers. In future studies, this aspect of the framework could be improved by investigating other methodologies for simplified collapse fragility curve estimation proposed in the literature or by developing new ones.
- Although the validation of the proposed frameworks' results conducted using time history analyses and the FEMA P-58 methodology showed promising results, more archetype buildings should be used to provide a comprehensive validation of the framework and to investigate the results of the framework for different structural typologies, non-structural element populations and building owners' profiles.
- Performance was expressed in the two proposed frameworks in terms of direct economic losses (Replacement Cost, Repair cost). However, future research could extend this study to indirect losses due to repair time for a more comprehensive prioritization of non-structural upgrades.
- Future research could take advantage of the two frameworks' ease of application to develop guidelines by analyzing a large population of structures with different structural behaviour, occupancy, non-structural elements and building owner conditions. These guidelines could be used by practitioners to have insight on non-structural upgrade prioritization with minimal resource investment.
- In the non-structural upgrade assessment framework, two parameters were proposed to assess the impact of non-structural upgrade in the first three steps of the framework, when a reduced amount of input data is required. These parameters are the maximum potential seismic loss reduction ΔL_{max} produced by a non-structural upgrade for a given earthquake scenario and the $\Delta L_{max}/UC$ ratio between the maximum potential loss reduction ΔL_{max} and the upgrade cost UC of implementing the upgrade. In the first steps of the framework, a threshold must

be assumed for these two parameters in order to eliminate non-structural upgrades that would not significantly contribute to reducing seismic losses or whose implementation cost is too high compared to the benefit of the upgrade. For the case study application illustrated in this thesis, 0.1% of the building cost and 0.5 were assumed as thresholds for ΔL_{max} and $\Delta L_{max}/UC$ to illustrate the framework application. However, future investigations on these parameters might be conducted to provide recommendations and guidance to users of the framework on which thresholds to assume depending on their intent.

- The building owner profile is characterized in the non-structural upgrade assessment framework by the definition of two parameters, which are the occupancy time and the internal rate of return. For the framework's application discussed in this thesis, an expected occupancy time of 40 years and an internal rate of return of 4% were assumed. Future research could extend this study to different owner profiles to investigate the impact of the owner profile on the prioritization of non-structural upgrades.

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APPENDIX A. ILLUSTRATIVE EXAMPLE OF AGGREGATE DAMAGE CALCULATION IN A FEMA P-58 ANALYSIS

The content of this appendix is part of the following journal publication: Banihashemi, A., Miliziano, A., Zsarnóczay, A., Wiebe, L., Filiatrault, A. [2023] "Consequences of consequence models: the impact of economies of scale on seismic loss estimates", accepted for publication in Earthquake Spectra.

This appendix presents four different damage aggregation methods, referred to as edge cases, that can be used to model economies of scale in the FEMA P-58-1 [2018] and in the frameworks proposed in this thesis. An illustrative example is discussed to highlight the influence of the four edge cases on repair cost calculation in a FEMA P-58 analysis.

Consequence functions for repair costs provided with the FEMA P-58 Methodology include consideration of economies of scale and operational efficiencies. Figure A. 1 illustrates a typical consequence function for repair costs.

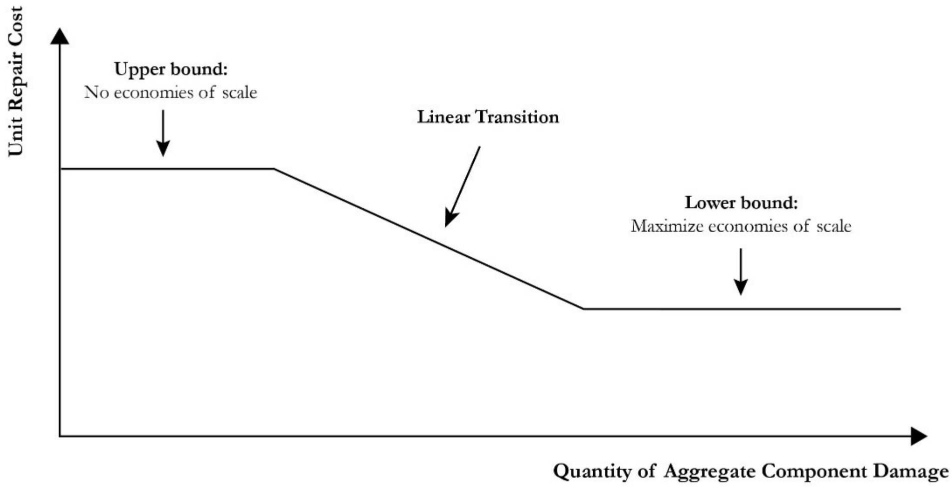


Figure A. 1 Typical consequence function for repair costs.

The median unit repair cost of each damaged component is a function of the aggregate quantity of damaged components. Depending on the quantities of damaged components, the median unit repair cost is estimated using the upper bound, the lower bound or the linear transition segment of a consequence function. When the upper bound is used, no economies of scale are considered; when the lower bound is used, all reasonable economies of scale are included; using the linear transition segment, the median unit repair cost is estimated considering a linear interpolation between the upper bound and lower bound median unit repair costs. To estimate the component unit repair cost in a FEMA P-58 analysis, a normal or a lognormal distribution with the median unit cost determined from the consequence function is then assumed.

This appendix focuses on the different interpretations that can be used to aggregate the quantity of damaged components used as input in a consequence function. Two decisions must be taken when aggregating damage: aggregate damage across floors or consider only the floor of interest, and aggregate damage across all damage states or consider only one damage state of interest at a time. Based on how these decisions are combined, one of the four different edge cases listed below is used:

1) "**All damage states, all floors"**: aggregate damage across all floors and from every damage state;

2) "**Individual damage state, all floors"**: aggregate damage across all floors but only from one damage state of interest at a time;

3) "**All damage states, individual floor"**: aggregate damage on the floor of interest from every damage state;

4) "**Individual damage state, individual floor"**: aggregate damage only on one floor and from one damage state of interest at a time.

The "All damage states, all floors edge case" is used both in the Performance Assessment Calculation Tool (PACT) developed within the FEMA P-58 Methodology and in the SP3 tool, which is a commercial tool that implements the FEMA P-58 methodology (Haselton Baker Risk Group, 2020). The assumption behind this edge case is that economies of scale are applied to all damaged items regardless of the location of the item and the severity of the damage. For instance, this assumption could be reasonable when considering economies of scale due to material costs. The "Individual damage state, all floors" edge case is employed by default by the current version of the Pelicun software (Zsarnóczay and Kourehpaz, 2021) developed by the NHERI Computational Modeling and Simulation Center (Deierlein et al., 2020). The assumption behind this second edge case is that economies of scale are applied to all damaged items in the same damage state regardless of their location. This assumption could be reasonable to apply economies of scale when different damage states do not share the same repair costs.

To illustrate the influence of the different edge cases on repair calculations, an illustrative example is presented below. Ten units of partition walls are assumed to be located on each floor of a two-storey building. The consequence functions provided in the FEMA P-58 documentation for wall component C1011.001b were used (Figure A.2). Two possible damage states (DS) can be obtained for the investigated component. The quantities of damaged items for the considered partition wall components on each floor and in each damage state obtained for a single realization are listed in Table A. 1.

Table A. 1. Illustrative example of the four edge cases to estimate unit repair cost of a component in two damage states (DS) across two floors in a building.

		Floor 1 DS1	Floor 2 DS1	Floor 1 DS2	Floor 2 DS2	Total component repair cost [$]
Quantity of damaged components [units]		3	5	2	3	
Case 1. All DS, all floors	Aggregate damage	13	13	13	13	22,218
	Cost per unit [$]	1071	1071	2730	2730	
	Cost [$]	3213	5355	5460	8190	
Case 2. Individual DS, all floors	Aggregate damage	8	8	5	5	44,355
	Cost per unit [$]	1626	1626	6269	6269	
	Cost [$]	4879	8132	12538	18807	
Case 3. All DS, individual floor	Aggregate damage	5	8	5	8	40,484
	Cost per unit [$]	2459	1626	6269	4146	
	Cost [$]	7378	8132	12538	12437	
Case 4. Individual DS, individual floor	Aggregate damage	3	5	2	3	61,178
	Cost per unit [$]	3015	2459	8392	7684	
	Cost [$]	9044	12297	16784	23053	

On the first floor, three units of partition walls are in Damage State 1 (DS1) and two units are in Damage State 2 (DS2). On the second floor, 5 units of partition walls are in DS1 and 3 units in DS2. The application of the four edge cases is illustrated in Figure A.2 and the total component repair costs obtained using the four cases are listed in Table A. 1. The lowest total repair cost is obtained using Case 1 (i.e. "All floors, all damage states" edge case). Using this edge case, economies of scale are applied to all damaged items regardless of their location and damage state. From Figure A. 2, it can be noticed that the aggregate damage estimated using Case 1 corresponds to the lower bound median unit repair cost in the consequence functions. Therefore, the maximum economies of scale are applied to all damaged units using this first edge case. The highest total repair cost, on the other hand, is obtained using Case 4 (i.e. "Individual damage state, Individual floor" edge case). The assumption behind this edge case is that economies of scale are applied only to damaged items on the same floor and in the same damage state. Using Case 4, the median unit repair costs are estimated from the linear transition segment of the consequence functions and the corresponding total repair cost is about three times the one obtained using Case 1. The total repair costs obtained using Case 2 (i.e., "Individual damage state, all floors") and Case 3 (i.e., "All damage states, individual floor") are in-between those calculated using Cases 1 and 4.

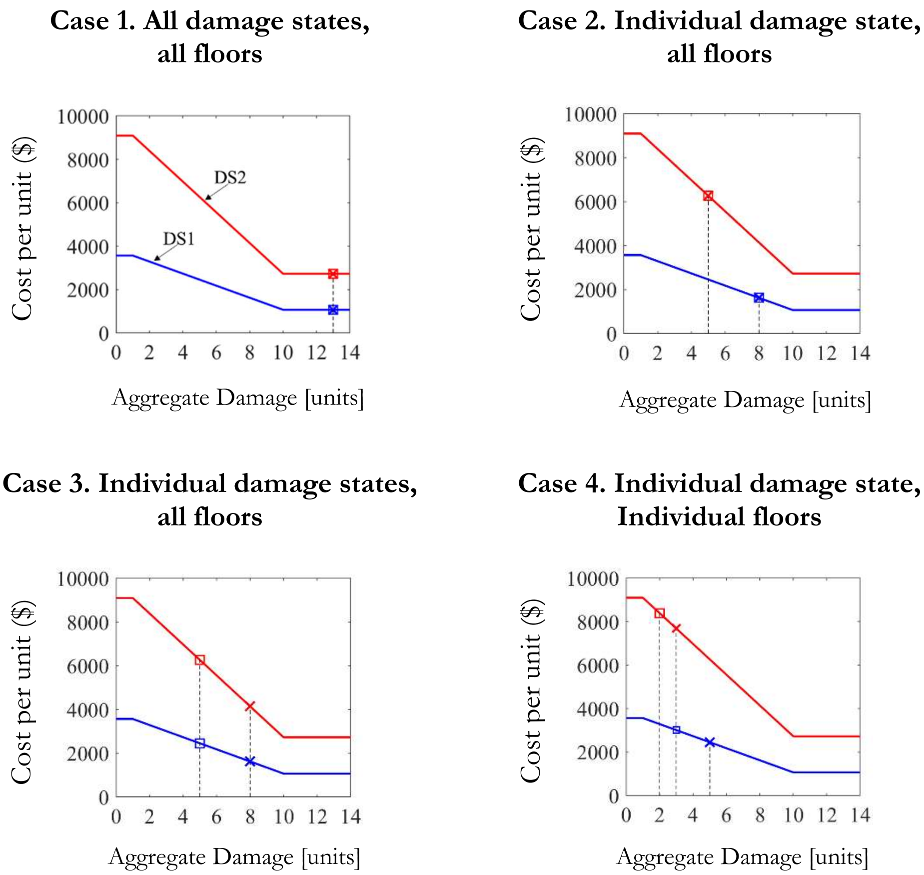


Figure A. 2 Illustrative example of the four edge cases to estimate unit repair cost of a component in two damage states (DS) across two floors in a building.

The illustrative example presented in this appendix showed that different edge cases for damage aggregation can yield substantially different estimates of repair costs for an individual component. However, several factors should be investigated to assess the overall impact of damage aggregation on seismic loss estimates. A comprehensive discussion on the impact of damage aggregation on seismic loss estimates is provided in Banihashemi et al. [2023], which also proposes a three-step evaluation strategy to allow engineers to quickly evaluate the potential impact of damage aggregation on a specific performance assessment.

REFERENCES

Banihashemi, A., Miliziano, A., Zsarnóczay, A., Wiebe, L., Filiatrault, A. [2023] "Consequences of consequence models: the impact of economies of scale on seismic loss estimates", accepted for publication in Earthquake Spectra.

FEMA P-58-1 [2018] "Seismic Performance Assessment of Buildings Volume 1 – Methodology (2nd Edit.)", FEMA P-58-1, Washington, D.C.: Federal Emergency Management Agency.

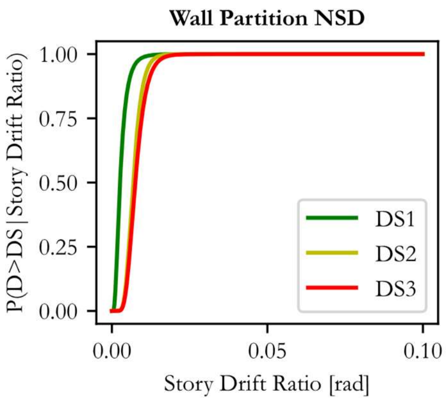
Haselton Baker Risk Group [2020] "Seismic Performance Prediction Program (SP3)", Retrieved from www.hbrisk.com.

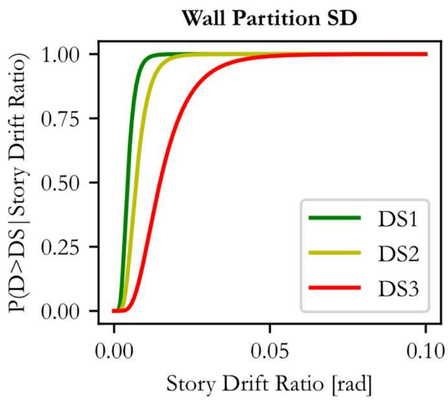
Zsarnóczay A., Deierlein G. G. [2020] "PELICUN - A Computational Framework for Estimating Damage, Loss and Community Resilience". *17th World Conference on Earthquake Engineering. Japan.*

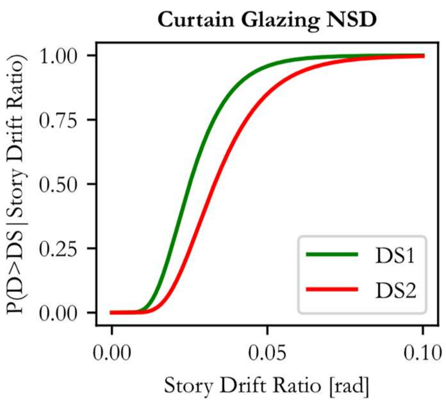
APPENDIX B. FRAGILITY FUNCTIONS ASSUMED IN CHAPTERS 4 AND 6

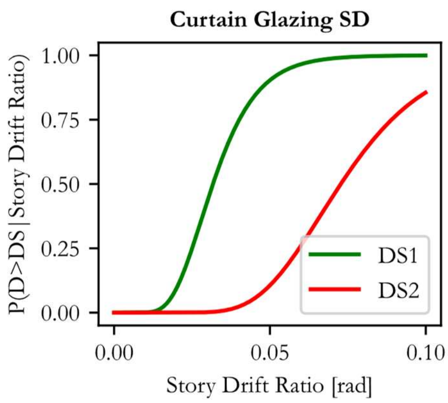
NSD = Not Seismically Designed

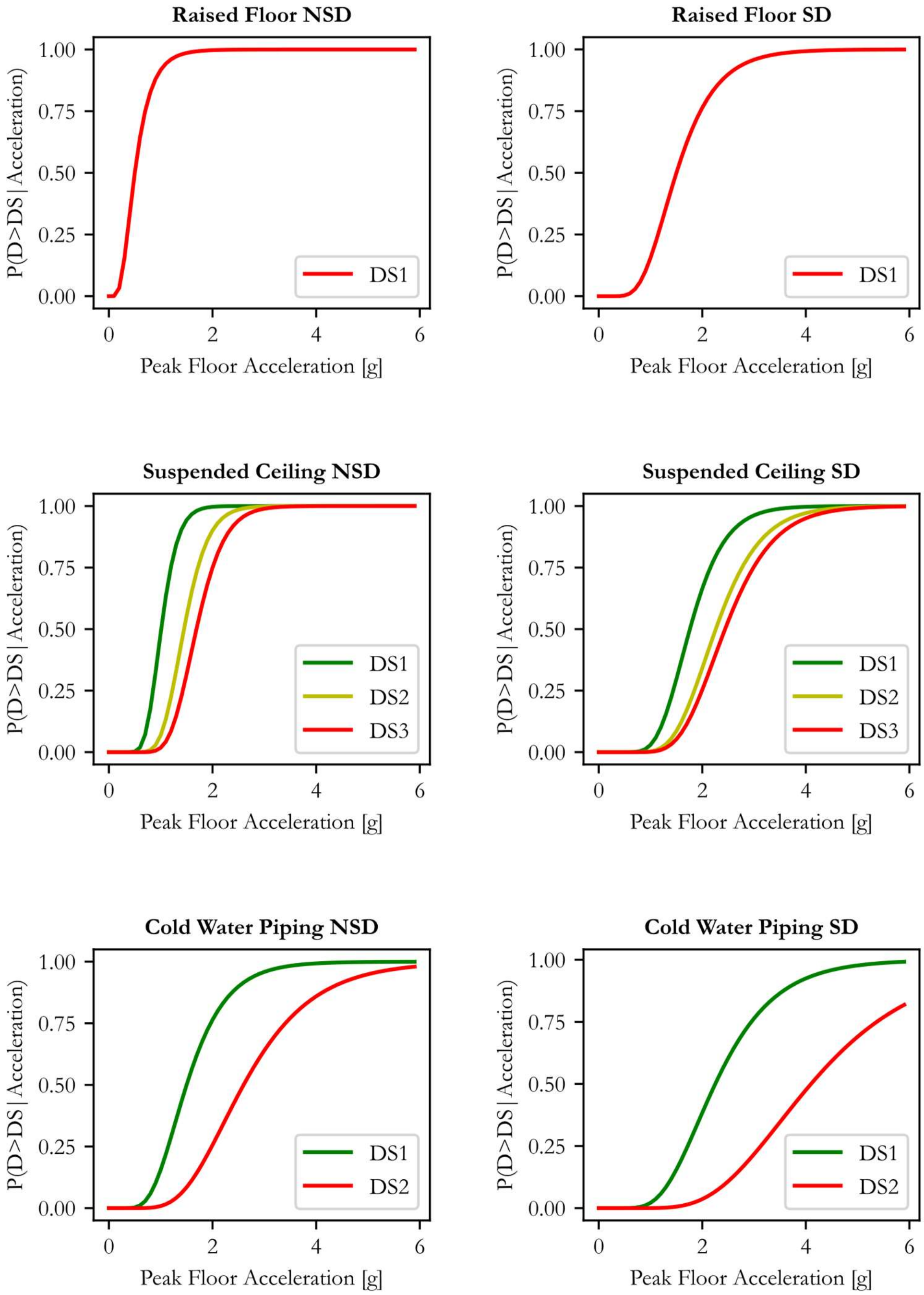
SD = Seismically Designed

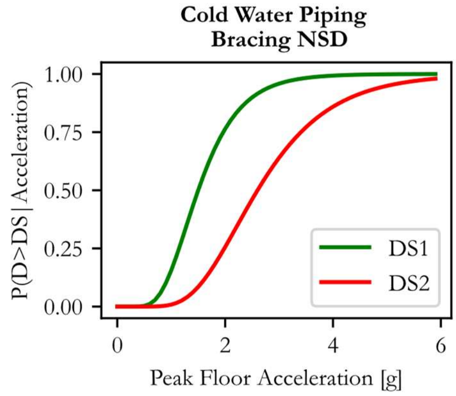


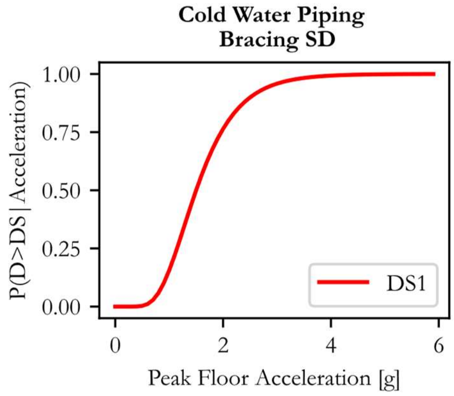


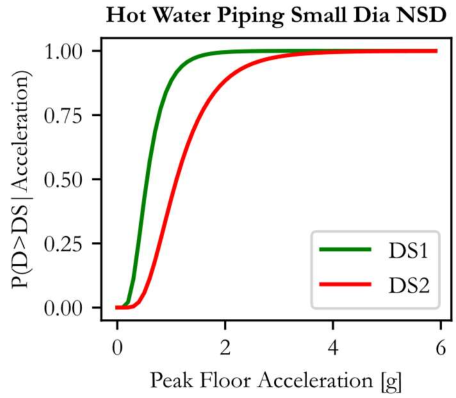





Raised Floor NSD
Raised Floor SD
Suspended Ceiling NSD
Suspended Ceiling SD
Cold Water Piping NSD
Cold Water Piping SD
P(D>DS|Acceleration)
Peak Floor Acceleration [g]
0.00
0.25
0.50
0.75
1.00
0
2
4
6
DS1
DS2
DS3


Cold Water Piping
Bracing NSD
P(D>DS | Acceleration)
1.00
0.75
0.50
0.25
0.00
0
2
4
6
Peak Floor Acceleration [g]
DS1
DS2


Cold Water Piping
Bracing SD
P(D>DS | Acceleration)
1.00
0.75
0.50
0.25
0.00
0
2
4
6
Peak Floor Acceleration [g]
DS1


Hot Water Piping Small Dia NSD
P(D>DS | Acceleration)
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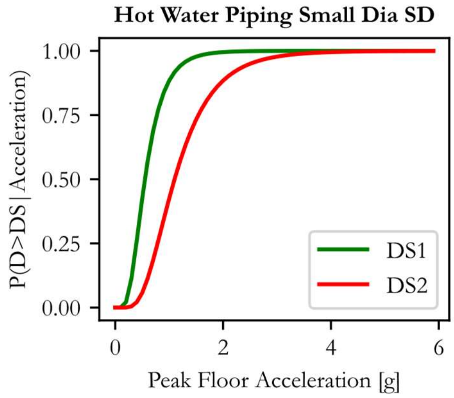
0.75
0.50
0.25
0.00
0
2
4
6
Peak Floor Acceleration [g]
DS1
DS2


Hot Water Piping Small Dia SD
P(D>DS | Acceleration)
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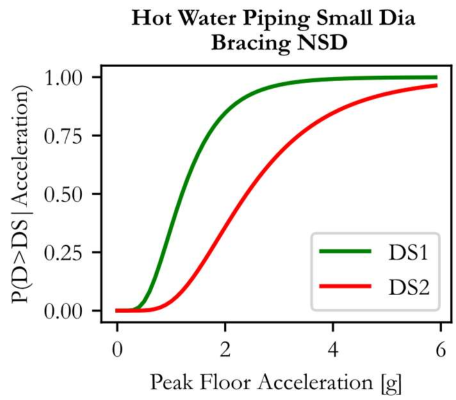
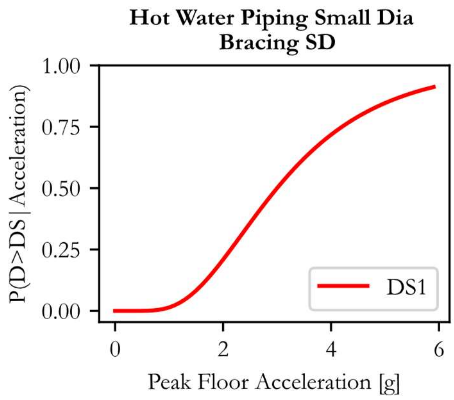
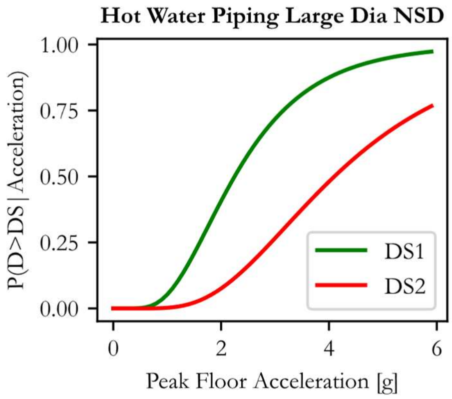
0.75
0.50
0.25
0.00
0
2
4
6
Peak Floor Acceleration [g]
DS1
DS2


Hot Water Piping Small Dia
Bracing NSD
P(D>DS | Acceleration)
1.00
0.75
0.50
0.25
0.00
0
2
4
6
Peak Floor Acceleration [g]
DS1
DS2


Hot Water Piping Small Dia
Bracing SD
P(D>DS | Acceleration)
1.00
0.75
0.50
0.25
0.00
0
2
4
6
Peak Floor Acceleration [g]
DS1


Hot Water Piping Large Dia NSD
P(D>DS|Acceleration)
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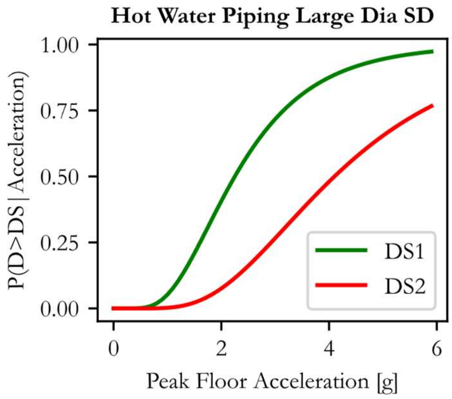
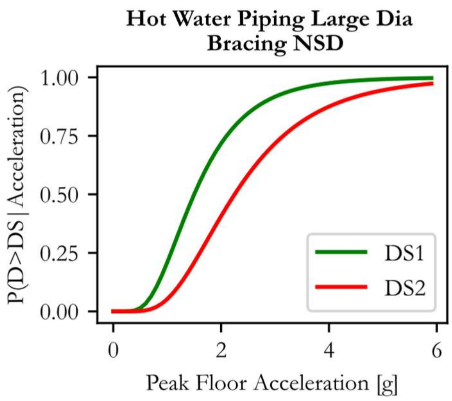
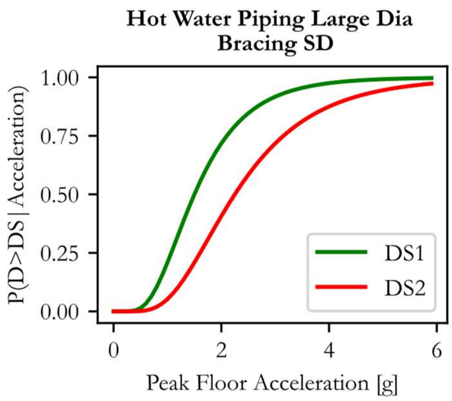
0.75
0.50
0.25
0.00
0
2
4
6
Peak Floor Acceleration [g]
DS1
DS2


Hot Water Piping Large Dia SD
P(D>DS|Acceleration)
1.00
0.75
0.50
0.25
0.00
0
2
4
6
Peak Floor Acceleration [g]
DS1
DS2


Hot Water Piping Large Dia Bracing NSD
P(D>DS|Acceleration)
1.00
0.75
0.50
0.25
0.00
0
2
4
6
Peak Floor Acceleration [g]
DS1
DS2


Hot Water Piping Large Dia Bracing SD
P(D>DS|Acceleration)
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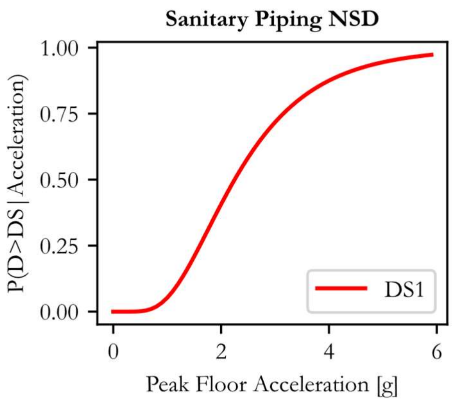
0.50
0.25
0.00
0
2
4
6
Peak Floor Acceleration [g]
DS1
DS2


Sanitary Piping NSD
P(D>DS|Acceleration)
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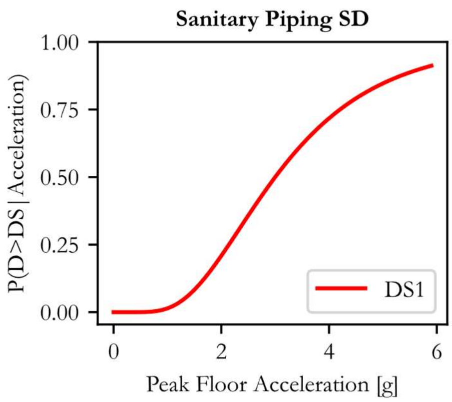
0.25
0.00
0
2
4
6
Peak Floor Acceleration [g]
DS1


Sanitary Piping SD
P(D>DS|Acceleration)
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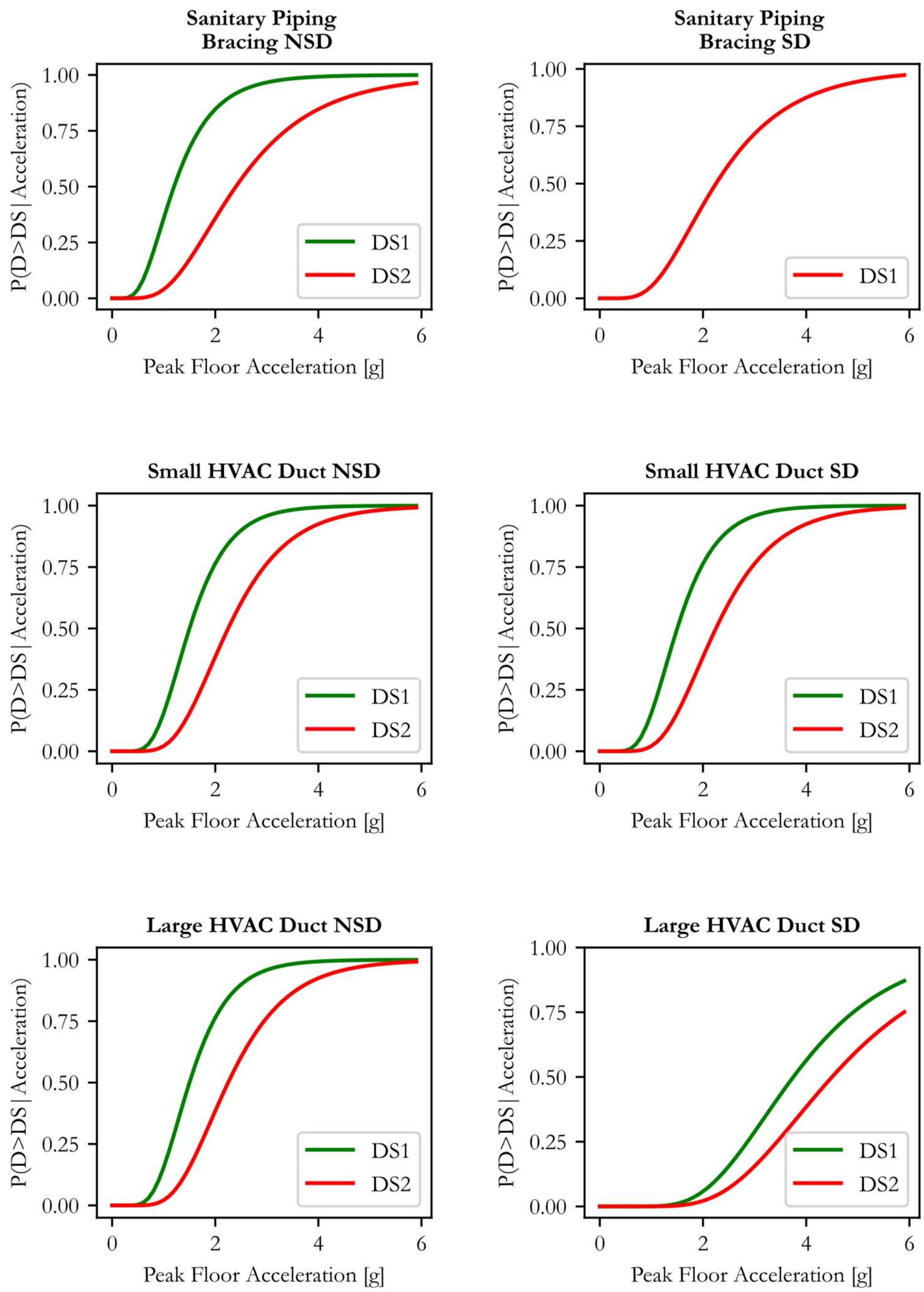
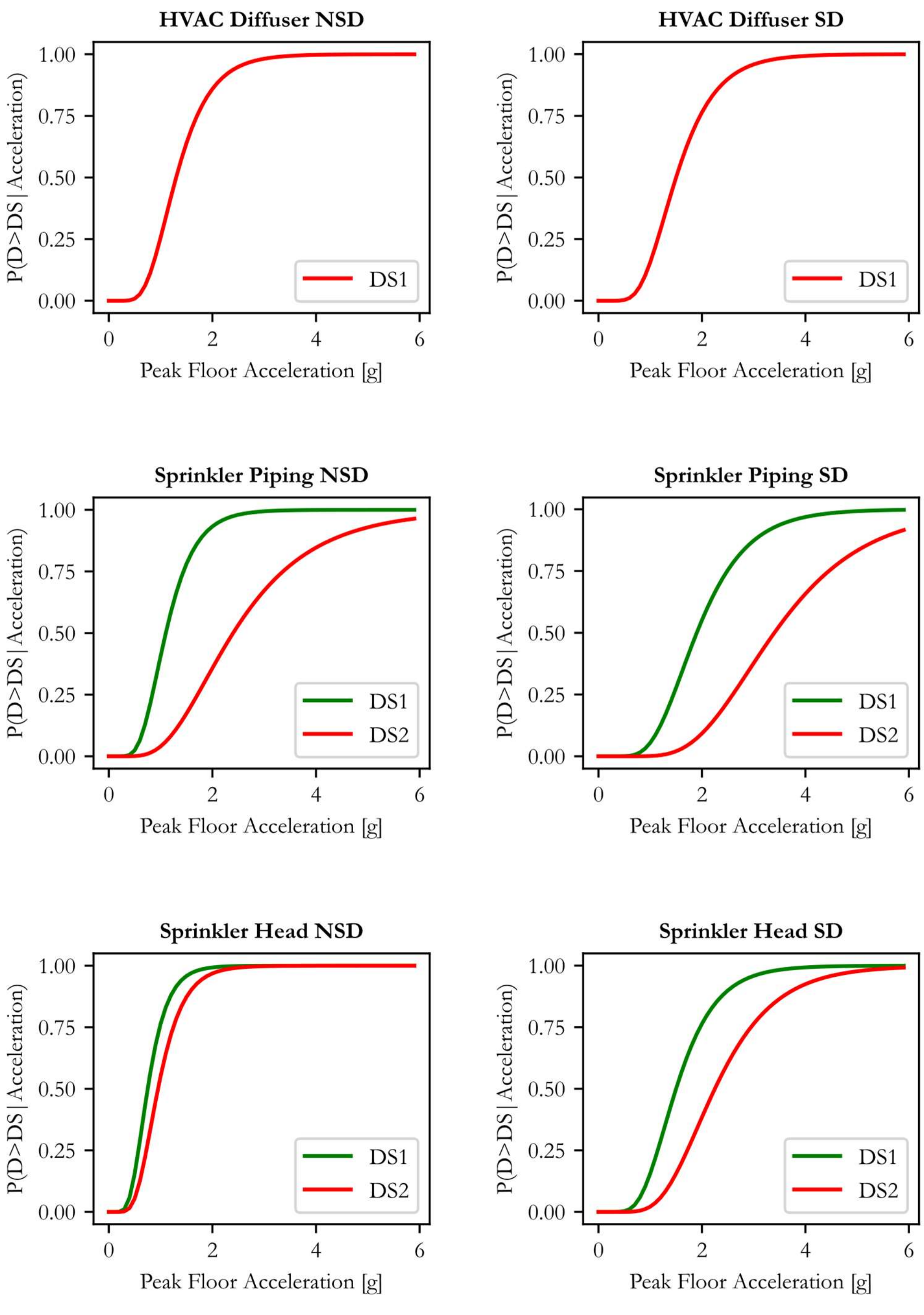
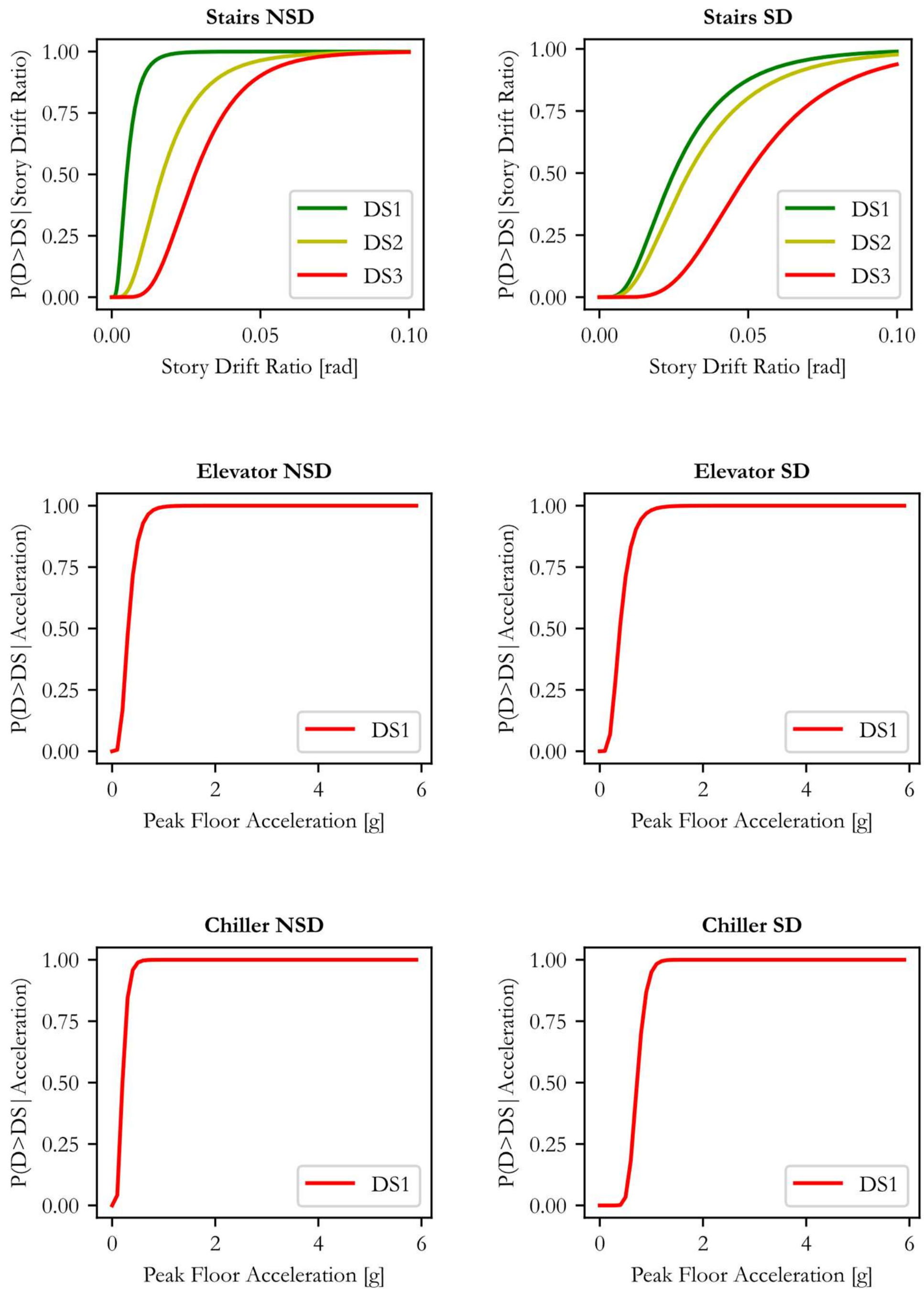
0.75
0.50
0.25
0.00
0
2
4
6
Peak Floor Acceleration [g]
DS1


Sanitary Piping Bracing NSD
Sanitary Piping Bracing SD
Small HVAC Duct NSD
Small HVAC Duct SD
Large HVAC Duct NSD
Large HVAC Duct SD
P(D>DS | Acceleration)
Peak Floor Acceleration [g]
DS1
DS2
0.00
0.25
0.50
0.75
1.00
0
2
4
6


HVAC Diffuser NSD
HVAC Diffuser SD
Sprinkler Piping NSD
Sprinkler Piping SD
Sprinkler Head NSD
Sprinkler Head SD
P(D>DS | Acceleration)
Peak Floor Acceleration [g]
DS1
DS2
0.00
0.25
0.50
0.75
1.00
0
2
4
6


Stairs NSD
Stairs SD
P(D>DS|Story Drift Ratio)
Story Drift Ratio [rad]
DS1
DS2
DS3
Elevator NSD
Elevator SD
Chiller NSD
Chiller SD
P(D>DS|Acceleration)
Peak Floor Acceleration [g]
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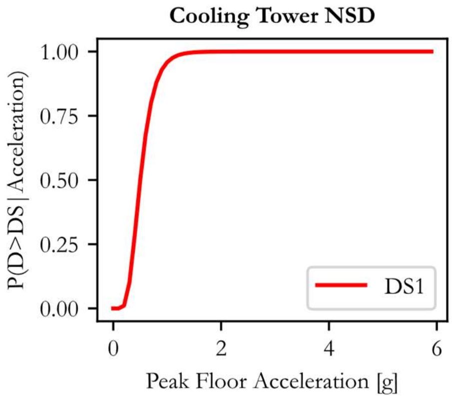
0.75
0.50
0.25
0.00
0.05
0.10
0
2
4
6


Cooling Tower NSD
P(D>DS|Acceleration)
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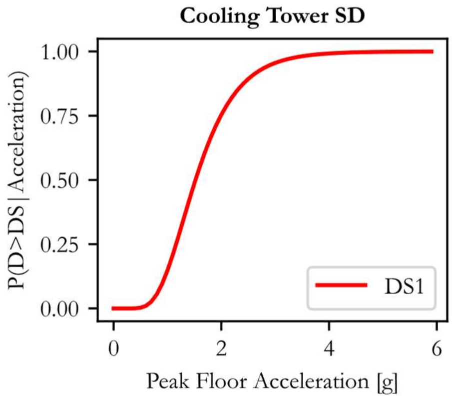
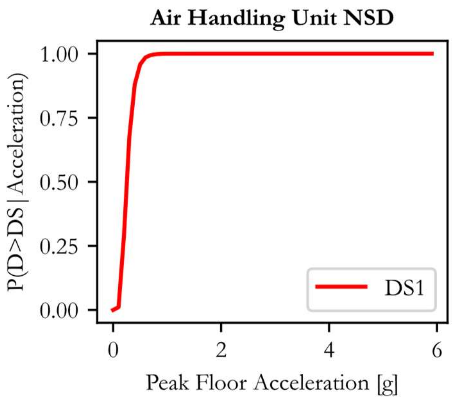
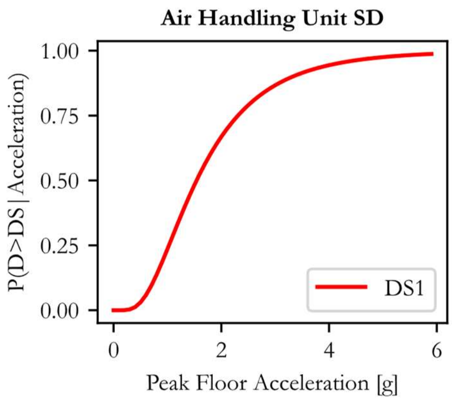
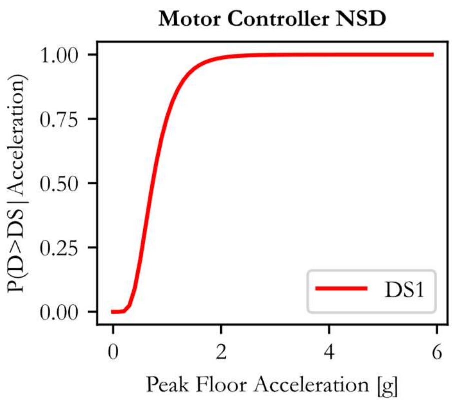
0.75
0.50
0.25
0.00
DS1
0
2
4
6
Peak Floor Acceleration [g]


Cooling Tower SD
P(D>DS|Acceleration)
1.00
0.75
0.50
0.25
0.00
DS1
0
2
4
6
Peak Floor Acceleration [g]


Air Handling Unit NSD
P(D>DS|Acceleration)
1.00
0.75
0.50
0.25
0.00
DS1
0
2
4
6
Peak Floor Acceleration [g]


Air Handling Unit SD
P(D>DS|Acceleration)
1.00
0.75
0.50
0.25
0.00
DS1
0
2
4
6
Peak Floor Acceleration [g]


Motor Controller NSD
P(D>DS|Acceleration)
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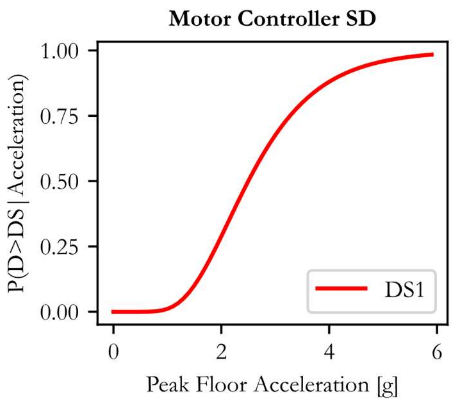
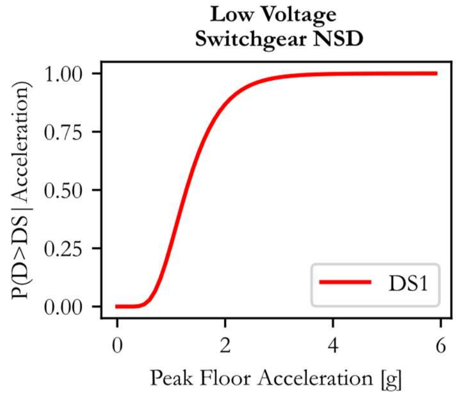
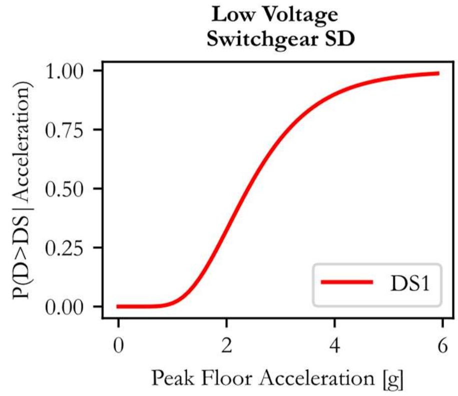
0.75
0.50
0.25
0.00
DS1
0
2
4
6
Peak Floor Acceleration [g]


Motor Controller SD
P(D>DS|Acceleration)
1.00
0.75
0.50
0.25
0.00
DS1
0
2
4
6
Peak Floor Acceleration [g]


Low Voltage
Switchgear NSD
P(D>DS | Acceleration)
1.00
0.75
0.50
0.25
0.00
DS1
0
2
4
6
Peak Floor Acceleration [g]


Low Voltage
Switchgear SD
P(D>DS | Acceleration)
1.00
0.75
0.50
0.25
0.00
DS1
0
2
4
6
Peak Floor Acceleration [g]

APPENDIX C. NON-STRUCTURAL UPGRADE ASSESSMENT TOOL - EXCEL IMPLEMENTATION

This appendix presents an Excel tool developed as an implementation of the non-structural upgrade assessment framework introduced in Chapter 3. The tool can be downloaded using the QR code at the end of this appendix. The downloadable Excel file is intended as a first version of the tool, which is expected to be improved in the future to include possible updates and required bug fixes.

The tool is composed by the following groups of Excel spreadsheets:

- **Notes.** This group comprises only one Excel Spreadsheet named "Notes" containing bibliographical information on the tool.
- **Input.** This group contains only one Excel Spreadsheet named "Input" in which users must specify all inputs required to apply the four steps of the framework.
- **Step Application.** This group is composed of six Excel Spreadsheets to apply each step of the framework. The first step application spreadsheet is named "Step 0" and it can be used to estimate non structural element quantities. The other Excel Spreadsheets are named "Step 1", "Step 2", "Step 3a", "Step 3b" and "Step 4". Each of these spreadsheets can be used to run a step of the framework. To facilitate its implementation, Step 3 is divided into two Excel spreadsheets ("Step 3a" and "Step 3b").
- **FEMA P-58 Database**. To facilitate the use of the tool, fragility and consequence functions in the FEMA P-58 database have been included in the three following Excel spreadsheets: "FEMA P-58 PERFORMANCE DATA", "FEMA P-58 COST DATA" and "Fragility Database". These excel spreadsheets are part of the FEMA P-58-3 [2018], which consists of a series of electronic products to assist engineers in assessing seismic performance of buildings and in understanding the technical basis of the FEMA P-58 methodology. Users of the tool don't need to interact with these Excel Spreadsheets to apply the framework. However, they can access these excel spreadsheets, add new components and manually update the FEMA P-58 fragility and consequence functions.
- **Step Calculation.** The framework calculations are implemented in the following five Excel Spreadsheets: "STEP 0 – Calculation", "STEP 1 – Calculation", "STEP 2 – Calculation", "STEP 3 – Calculation" and "STEP 4 – Caluculation". Users don't need to interact with these spreadsheets in order to use the tool.

In the next paragraphs, the content of the spreadsheets that require users' interaction will be discussed in detail and guidelines on how to use them will be provided.

C.1 INPUT SPREADSHEET

All input data required to apply the framework must be inserted in the “Input Data” Excel Spreadsheet. Cells with the same colour correspond to input data required for the same step of the framework. Figure C. 1 shows the colours used for the four steps of the framework.

Input Data - STEP 1
Input Data - STEP 2
Input Data - STEP 3
Input Data - STEP 4

Figure C. 1 Colours used to indicate input data for different steps of the framework.

Input Data – STEP 1. The first step requires the lowest amount and level of sophistication of input data, and it can be run without performing any structural analysis. Figure C. 2 illustrates all the data that should be specified by users in order to apply Step 1. General information on building occupancy, number of floors and building cost must be specified. A maximum number of 12 floors can be assessed using the current version of the tool. “Floor 1” in the input data table corresponds to the ground floor of the building.The maximum loss reduction threshold $\Delta L_{,max,\ threshold}$ should be also specified. This parameter is used to exclude non-structural upgrades that would not significantly contribute to seismic loss reduction. Oher input data required to apply Step 1 are the floor area and the non-structural element set. Information on the non-structural elements must be input in a table which contain the following columns:

- Component Name. In this column, non-structural component names should be specified by users.
- ID Before Upgrade. This is the column in which users must specify the ID of the not-seismically designed configuration of each component. The ID can be selected from a drop-down menu which includes all components available in the FEMA P-58 database, as defined in the FEMA P-58 perfomance data. Fragility functions and consequence functions associated with the specified ID are used to perform the analysis in the calculation spreadsheets.
- ID After Upgrade. This is the column in which users must specify the ID of the seismically designed configuration of each component.
- Component type for quantity estimation. In this column, users must specify the component type corresponding to each investigated component. Component types can be selected from a drop-down menu. This information is used in the tool to estimate component quantities.
- % Quantity Direction 1. Components must be assigned to an appropriate direction between direction 1, direction 2 and non-directional, as is done in the FEMA P-

58 methodology (FEMA P-58-1, 2018). This choice depends on the component demand parameter. For instance, acceleration-sensitive components are typically assigned independent of direction (non-directional) because the predictive demand parameter is peak floor acceleration. In the "% Quantity Direction 1" column, users must specify the percentage of component quantity in direction 1. For example, for acceleration-sensitive non-structural components this percentage is typically zero, while for drift-sensitive it may be 100% or less, depending on how components are distributed in the directions 1 and 2.

- % Quantity Direction 2. In this column, users must specify the percentage of component quantity in direction 2.
- % Quantity Non-Directional. In this column, users must specify the percentage of component quantity assigned to be independent from the direction.
- Floors with component. This part of the table can be used to specify the location of each component. For each component and for each floor, an Excel cell needs to be filled by users by selecting from a drop-down menu "yes" or "no". A component is assumed to be located at all floors for which "yes" is selected.
-

Occupancy	OFFICE
Number of floors	7
Building Cost [$]	12509332.76
∆Lmax Threshold [% Building Cost]	0.10

	Floor 1	Floor 2	Floor 3
Floor Area [SF]	8640.00	8640.00	8640.00

Component Name	ID Before Upgrade	ID After Upgrade	Component Type for Quantity Estimation
HVAC duct, Small Area	D3041.011a	D3041.011c	HVAC - HVAC Ducts – less than 6 sq. feet
HVAC duct, Large Area	D3041.012a	D3041.012d	HVAC - HVAC Ducts – 6 sq. feet for larger
HAVC diffuser	D3041.031a	D3041.032d	HVAC - HVAC in-line Drops & Diffusers
Sprinkler Piping	D4011.021a	D4011.024a	Fire Protection - Sprinkler Piping
Sprinkler head	D4011.031a	D4011.053a	Fire Protection - Sprinkler Drops
Elevators	D1014.012	D1014.011	Elevators
Chiller	D3031.011c	D3031.013h	HVAC - Chiller capacity
Cooling Tower	D3031.021c	D3031.023h	HVAC - Cooling Tower capacity
Air Handling Unit	D3052.011d	D3052.013k	HVAC - Air Handling Units
Motor Control	D5012.013a	D5012.013c	Electrical - Motor Control Centers
Low Voltage Switchgear	D5012.021b	D5012.023e	Electrical - Wall mounted switchgear

Component Name	% Quantity Direction 1	% Quantity Direction 2	% Quantity Non Directional	Floor with component? Non Directional Floor1	Floor2	Floor3
HVAC duct, Small Area	0%	0%	100%	YES	YES	YES
HVAC duct, Large Area	0%	0%	100%	YES	YES	YES
HAVC diffuser	0%	0%	100%	YES	YES	YES
Sprinkler Piping	0%	0%	100%	YES	YES	YES
Sprinkler head	0%	0%	100%	YES	YES	YES
Elevators	0%	0%	100%	YES	NO	NO
Chiller	0%	0%	100%	NO	NO	NO
Cooling Tower	0%	0%	100%	NO	NO	NO
Air Handling Unit	0%	0%	100%	NO	NO	NO
Motor Control	0%	0%	100%	NO	NO	NO
Low Voltage Switchgear	0%	0%	100%	YES	YES	YES

Figure C. 2 Input data required to apply Step 1.

Input Data – STEP 2. Step 2 requires as additional input an estimate of non-structural upgrade costs *UC*. The $\Delta L_{max}/UC$ ratio between the maximum potential loss reduction and the upgrade cost is used as a parameter to assess non-structural upgrade in Step 2. A $\Delta L_{max}/UC$ ratio threshold must be specified by users of the framework in order to perform Step 2. Input data required to apply the second step are illustrated in Figure C. 3.

ΔL max / Upgrade Cost Threshold	0.50

Component Name	Upgrade Cost [% Building Cost]
HVAC duct, Small Area	0.71
HVAC duct, Large Area	0.56
HAVC diffuser	1.28
Sprinkler Piping	1.49
Sprinkler head	1.00
Elevators	0.22
Chiller	0.68
Cooling Tower	0.68
Air Handling Unit	5.85
Motor Control	0.03
Low Voltage Switchgear	0.08

Figure C. 3 Input data required to apply Step 2.

Input Data – STEP 3. Step 3 needs as additional input the maximum expected values of inter-storey drift and peak floor acceleration for each direction of the investigated building (Figure C. 4).

Drift max Direction 1 [rad]	0.03
Drift max Direction 2 [rad]	0.03
PFA max Direction 1 [g]	2.32
PFA max Direction 2 [g]	2.32

Figure C. 4 Input data required to apply Step 3.

Input Data – STEP 4. Step 4 is the most refined and requires the following additional input data: the hazard curve at the site, structural response parameters for different earthquake intensities (average EDP values and collapse fragility), and parameters to perform the benefit-cost analysis (i.e. expected occupancy time of the building, internal rate of return). The input data required to apply Step 4 are illustrated in Figure C. 5. A maximum of 12 intensity levels can be assessed using the tool.

Number of intensities	6

				Floor 1				Floor 2			
				Direction 1		Dierction 2		Direction 1		Dierction 2	
Intensity	SaT1 [g]	annual frequency of exceedence	Drift Residual [rad]	Drift [rad]	PFA [g]	Drift [rad]	PFA [g]	Drift [rad]	PFA [g]	Drift [rad]	PFA [g]
1	0.11	0.0322	0.000	0.00	0.16	0.00	0.16	0.00	0.31	0.00	0.31
2	0.20	0.0139	0.000	0.01	0.29	0.01	0.29	0.01	0.46	0.01	0.46
3	0.61	0.0021	0.002	0.02	1.00	0.02	1.00	0.02	0.92	0.02	0.92
4	0.86	0.0010	0.005	0.03	1.53	0.03	1.53	0.02	1.13	0.02	1.13
5	1.26	0.0004	0.00	0.03	2.32	0.03	2.32	0.03	1.50	0.03	1.50
6	1.59	0.00	0.01	0.03	3.51	0.03	3.51	0.03	1.97	0.03	1.97

Collapse Fragility	
Median	1.64
Dispersion	0.55

Demolition Fragility	
Median Irreparable Residual Drift]	0.01
Dispersion	0.3

occupancy time	40
r	0.04
Benefit Cost Ratio Threshold	1

Figure C. 5 Input data required to apply Step 4.

C.2 STEP APPLICATION SPREADSHEETS

An excel spreadsheet is dedicated to the application of each step of the framework. Each step application spreadsheet contains two buttons to run the step and to reset it. The "Reset Step" button is enabled only when the "Run Step" button is clicked and, viceversa, after a step is run, the "Reset Step" button must be clicked to enable the "Run Step" button again. Once the "Reset Step" button of a step is clicked, all following steps are reset and must be run again. A brief description of the step application spreadsheets is provided in this section.

"Step 0" Spreadsheet. The "Step 0" spreadsheet can be used to estimate element quantities based on the normative quantities provided within the FEMA P-58-3 [2018]. When the "Step 0 Run" button is clicked, a table is filled with the component quantity estimates at each floor. A screen capture of the "Step 0" Spreadsheet after the "Step 0 Run" button is clicked is illustrated in Figure C. 6. After the quantity estimation is performed, the quantity estimates in the table can be also manually modified by users of the tool. However, if quantity estimates are modified, the subsequent steps are not automatically updated and must be rerun to provide results that are consistent with the modified input quantities.

Component Name	ID Before Upgrade	ID After Upgrade	Non Directional		
			Floor1	Floor2	Floor3
HVAC duct, Small Area	D3041.011a	D3041.011c	0.65	0.65	0.65
HVAC duct, Large Area	D3041.012a	D3041.012d	0.17	0.17	0.17
HAVC diffuser	D3041.031a	D3041.032d	7.78	7.78	7.78
Sprinkler Piping	D4011.021a	D4011.024a	1.73	1.73	1.73
Sprinkler head	D4011.031a	D4011.053a	0.78	0.78	0.78
Elevators	D1014.012	D1014.011	2.00	0.00	0.00
Chiller	D3031.011c	D3031.013h	0.00	0.00	0.00
Cooling Tower	D3031.021c	D3031.023h	0.00	0.00	0.00
Air Handling Unit	D3052.011d	D3052.013k	0.00	0.00	0.00
Motor Control	D5012.013a	D5012.013c	0.00	0.00	0.00
Low Voltage Switchgear	D5012.021b	D5012.023e	1.00	1.00	1.00

STEP0 Run

STEP0 Reset

Figure C. 6 Screen capture of the "Step 0" Spreadsheet.

"Step 1" Spreadsheet. Figure C. 7 shows the "Step 1" Excel spreadsheet as it appears after the step is run. The non-structural elements input in the "Input" spreadsheet appear in the first column of the "Step 1" spreadsheet. In the second column, the ΔL_{max} values calculated for each element are listed. These values are compared with the ΔL_{max} threshold specified by the user in the "Input" spreadsheet. Only the element upgrades with ΔL_{max} above the threshold will appear on the "Step 2" Excel spreadsheet. For these upgrades, the word "yes" appears in the column named "Pass Step 1?".

Component Name	ΔLmax [$]	Dlmax [% Building Cost]	DLmax Threshold [% Building Cost]	Pass Step 1 ?	Filter
Wall Partitions*	260216	2.08	0.10	YES	1
Curtain Wall Glazing*	169764	1.36	0.10	YES	1
Raised Access Floor	26258	0.21	0.10	YES	1
Suspended Ceilings	387941	3.10	0.10	YES	1
Cold Water Piping, Small Diameter - Piping Fragility	2535	0.02	0.10	NO	1
Cold Water Piping, Small Diameter - Bracing Fragility	7814	0.06	0.10	NO	1
Hot Wate Piping, Small Diameter - Piping Fragility	0	0.00	0.10	NO	1
Hot Wate Piping, Small Diameter - Bracing Fragility	15563	0.12	0.10	YES	1
Hot Wate Piping, Large Diameter - Piping Fragility	0	0.00	0.10	NO	1
Hot Wate Piping, Large Diameter - Bracing Fragility	0	0.00	0.10	NO	1
Sanitary Piping - Piping Fragility	4470	0.04	0.10	NO	1
Sanitary Piping - Bracing Fragility	21922	0.18	0.10	YES	1
HVAC duct, Small Area	0	0.00	0.10	NO	1
HVAC duct, Large Area	5585	0.04	0.10	NO	1
HAVC diffuser	16181	0.13	0.10	YES	1
Sprinkler Piping	9095	0.07	0.10	NO	1
Sprinkler head	1518	0.01	0.10	NO	1
Stairs*	80557	0.64	0.10	YES	1
Elevators	20800	0.17	0.10	YES	1
Chiller	237102	1.90	0.10	YES	1
Cooling Tower	119457	0.95	0.10	YES	1
Air Handling Unit	127952	1.02	0.10	YES	1
Motor Control	10535	0.08	0.10	NO	1
Low Voltage Switchgear	64737	0.52	0.10	YES	1

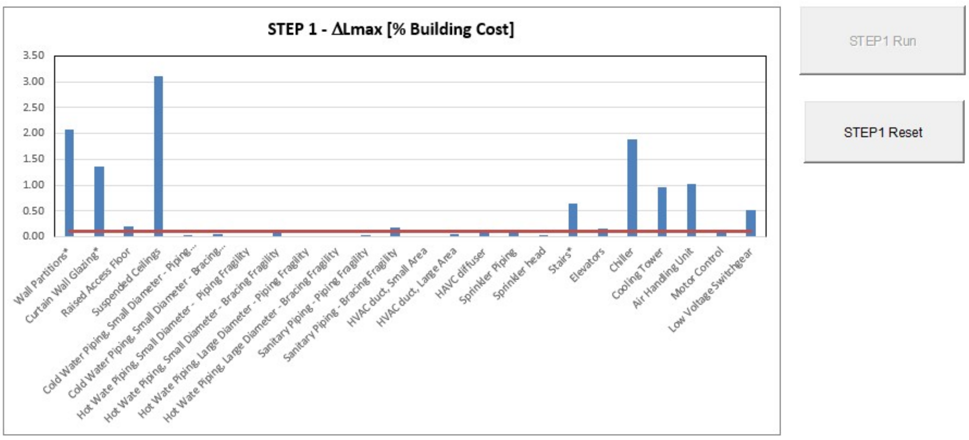


Figure C. 7 Screen capture of the "Step 1" Spreadsheet.

"Step 2" Spreadsheet. When running Step 2, for all the non-structural elements remaining after Step 1, the $\Delta L_{max}/UC$ ratio is calculated and compared with the $\Delta L_{max}/UC$ ratio threshold specified in the "Input" spreadsheet. Only the element with a $\Delta L_{max}/UC$ ratio above the threshold will appear on the "Step 3" spreadsheet. A screen capture showing the results of Step 2 in the "Step 2" spreadsheet is provided in Figure C. 8.

Component Name	Dlmax/UC	Dlmax/UC Threshold	Pass Step 2 ?	Filter
Wall Partitions*	0.68	0.50	YES	1
Curtain Wall Glazing*	0.21	0.50	NO	1
Raised Access Floor	0.75	0.50	YES	1
Suspended Ceilings	0.57	0.50	YES	1
Hot Wate Piping, Small Diameter - Bracing Fragility	0.14	0.50	NO	1
Sanitary Piping - Bracing Fragility	0.25	0.50	NO	1
HAVC diffuser	0.10	0.50	NO	1
Stairs*	1.85	0.50	YES	1
Elevators	0.74	0.50	YES	1
Chiller	2.78	0.50	YES	1
Cooling Tower	1.40	0.50	YES	1
Air Handling Unit	0.17	0.50	NO	1
Low Voltage Switchgear	6.17	0.50	YES	1

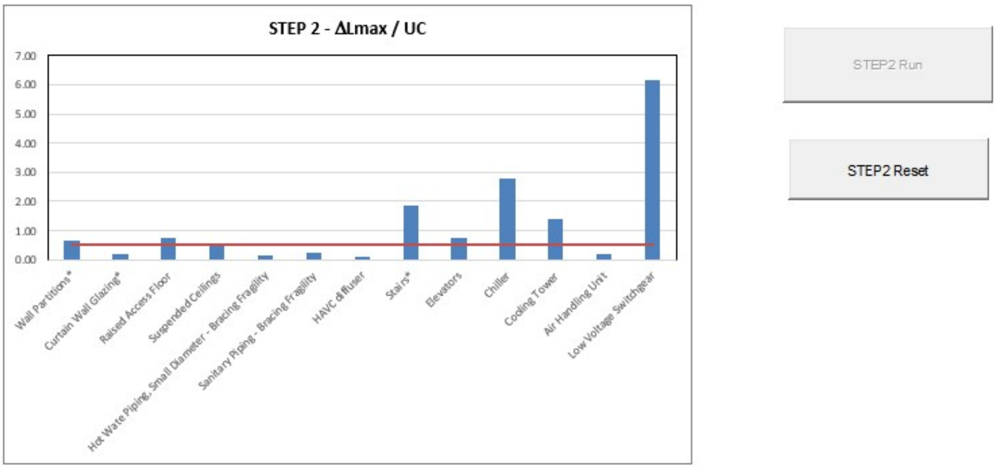


Figure C. 8 Screen capture of the "Step 2" Spreadsheet.

"Step 3a" and "Step 3b" Spreadsheets. To facilitate the implementation of Step 3 in the Excel tool, the third step of the framework was divided into two spreadsheets. The "Step 3a Run" button in the "Step 3a" Excel spreadsheet allows to calculate ΔL_{max} for an EDP range of interest adjusted based on the input maximum expected EDPs. The "Step 3b Run" button in the "Step 3b" Excel sheet allows to calculate $\Delta L_{max}/UC$ for the same EDP range of interest. A screen capture of the two spreadsheets to apply Step 3 is provided in Figure C. 9 and Figure C. 10. Only components with ΔL_{max} and $\Delta L_{max}/UC$ greater than the thresholds specified in the "Input" spreadsheet will appear on the "Step 4" Excel spreadsheet.

Component Name	Dlmax [$]	DLmax [% Building Cost]	DLmax Threshold [% Building Cost]	Pass Step 3a?	Filter
Wall Partitions*	240879.61	1.93	0.10	YES	1
Raised Access Floor	26257.63	0.21	0.10	YES	1
Suspended Ceilings	387941.38	3.10	0.10	YES	1
Stairs*	75162.11	0.60	0.10	YES	1
Elevators	20799.82	0.17	0.10	YES	1
Chiller	237102.33	1.90	0.10	YES	1
Cooling Tower	119456.78	0.95	0.10	YES	1
Low Voltage Switchgear	64737.04	0.52	0.10	YES	1

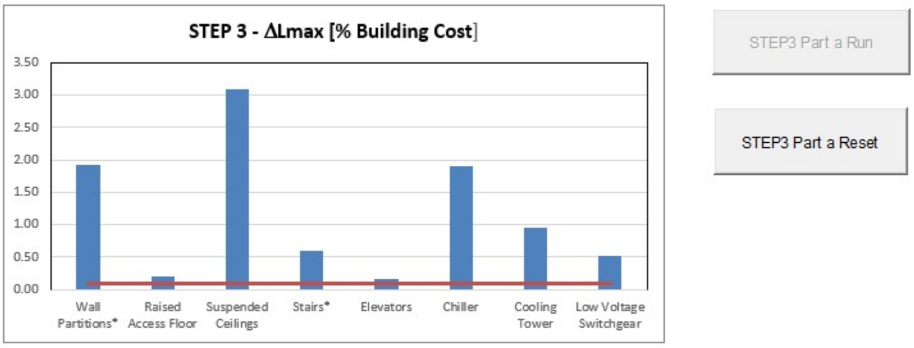


Figure C. 9 Screen capture of the "Step 3a" Spreadsheet.

Component Name	Dlmax/UC	Dlmax/UC Threshold	Pass Step 3 - Part 2 ?	Filter
Wall Partitions*	0.63	0.50	YES	1
Raised Access Floor	0.75	0.50	YES	1
Suspended Ceilings	0.57	0.50	YES	1
Stairs*	1.73	0.50	YES	1
Elevators	0.74	0.50	YES	1
Chiller	2.78	0.50	YES	1
Cooling Tower	1.40	0.50	YES	1
Low Voltage Switchgear	6.17	0.50	YES	1

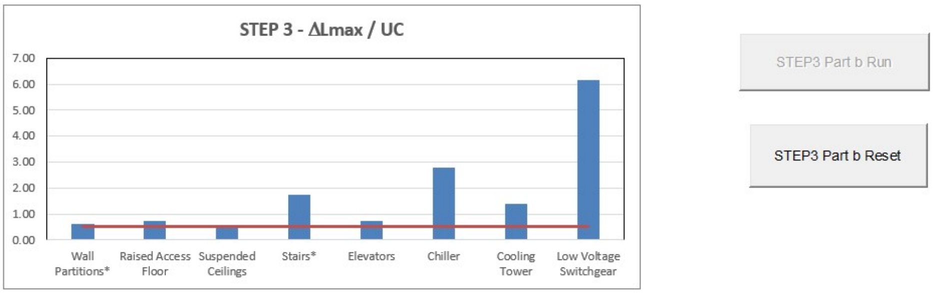


Figure C. 10 Screen capture of the "Step 3b" Spreadsheet.

"Step 4" Spreadsheet. Figure C. 11 shows a screen capture of the "Step 4" spreadsheet. When running the "Step4 Run" button on the "Step 4" Excel sheet, elements are prioritized according to their benefit-cost ratio. The figure generated in this step shows the benefit-cost ratio of all components remaining after Step 3.

Component Name	∆EAL	BCR	BCR Threshold	Pass Step 4?	Filter
Wall Partitions*	4644.58	0.24	1	NO	1
Raised Access Floor	443.14	0.25	1	NO	1
Suspended Ceilings	553.15	0.02	1	NO	1
Stairs*	353.04	0.16	1	NO	1
Elevators	360.44	0.25	1	NO	1
Chiller	**4929.65**	**1.14**	**1**	**YES**	**1**
Cooling Tower	2978.51	0.69	1	NO	1
Low Voltage Switchgear	173.90	0.33	1	NO	1

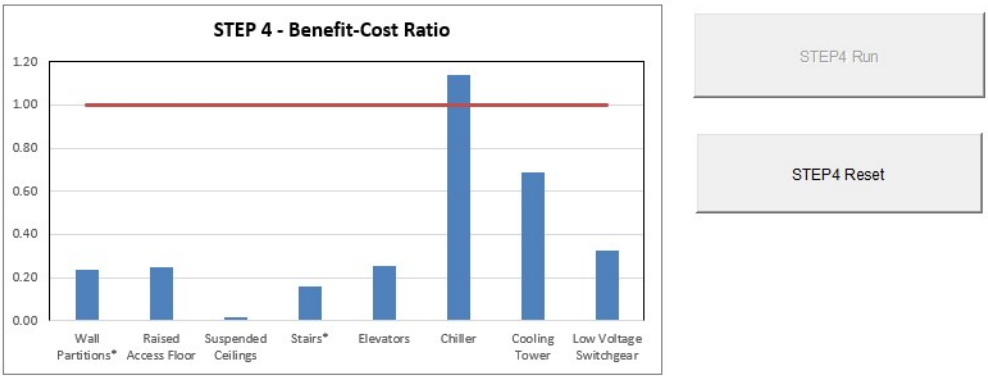


Figure C. 11 Screen capture of the "Step 4" Spreadsheet.

REFERENCES

FEMA P-58-1 [2018] "Seismic Performance Assessment of Buildings Volume 1 – Methodology (2nd Edit.)", FEMA P-58-1, Washington, D.C.: Federal Emergency Management Agency.

FEMA P-58-3 [2018] "Seismic Performance Assessment of Buildings Volume 3 – Supporting Electronic Materials and Background Documentation (Third Edition), FEMA P-58-3, Washington, D.C.: Federal Emergency Management Agency.

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(https://qr.page/g/4bMXpwqEAA)